FOUNDATIONS, ABUTMENTS
AND FOOTINGS

FOUNDATIONS, ABUTMENTS AND FOOTINGS

COMPILED BY A STAFF OF SPECIALISTS

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FIRST EDITION
SIXTH IMPRESSION

McGRAW-HILL BOOK COMPANY, INC.
NEW YORK: 370 SEVENTH AVENUE
LONDON: 6 & 8 BOUVERIE ST., E. C. 4
1923

PRINTED IN THE UNITED STATES OF AMERICA

THE MAPLE PRESS - YORK PA

PREFACE

This volume is one of a series designed to provide the engineer and the student with a reference work covering thoroughly the design and construction of the principal kinds and types of modern civil engineering structures. An effort has been made to give such a complete treatment of the elementary theory that the books may also be used for home study.

The titles of the six volumes comprising this series are as follows:

Foundations, Abutments and Footings
Structural Members and Connections
Stresses in Framed Structures
Steel and Timber Structures
Reinforced Concrete and Masonry Structures
Movable and Long-span Steel Bridges

Each volume is a unit in itself, as references are not made from one volume to another by section and article numbers. This arrangement allows the use of any one of the volumes as a text in schools and colleges without the use of any of the other volumes.

Data and details have been collected from many sources and credit is given in the body of the books for all material so obtained. A few chapters, however, throughout the six volumes have been taken without special mention, and with but few changes, from Hool and Johnson's Handbook of Building Construction.

The Editors-in-Chief wish to express their appreciation of the spirit of cooperation shown by the Associate Editors and the Publishers. This spirit of cooperation has made the task of the Editors-in-Chief one of pleasure and satisfaction.

G. A. H.
W. S. K.

MADISON, WIS.
April, 1923.

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FOUNDATIONS, ABUTMENTS AND FOOTINGS

SECTION 1

SOIL INVESTIGATION

TESTS TO DETERMINE SOIL CONDITIONS

BY R. C. SMITH

Sub-surface investigations should precede the designing of foundations in order to determine the existing soil conditions, and the safe bearing power of the soil.[1]

1. Test Pits.—There are a number of methods of determining to a more or less accurate degree the soil stratification at any given location. The oldest and best method, of course, is by actual excavation in test pits into the earth. By this method the stratification is exposed for examination, and the conditions to be met in actual building operations are seen in their true relations. The firmness of the material, its water content, tendency to run or cave, and the extent to which sheeting and bracing of banks will be necessary becomes evident as the excavation in the pit progresses.

Test pits are made in the same way that mine or tunnel shafts are sunk, or as open wells are dug in sections where open wells are used. In general, where the pit is more than 10 or 15 ft. in depth, and often where only 6 or 8 ft. in depth, the walls of the pit will have to be sheeted, and some form of bracing used.

In test pit examinations, observations should be made from the materials in situ, and not made up entirely from an examination of the excavated material some time after it has been taken from the pit. The material as taken from the pit should be piled in the order taken out, and sufficiently scattered to allow

[1] For characteristics of soils, etc., see Appendix *A*.

thorough examination. A better idea of the relative amounts of the several kinds of materials will be gleaned in this way than will be had from the exposed sides of the pit, but the hardness of the various strata will not be so evident.

On work involving large quantities of excavation where but little is known of the earth stratification, test pit examinations should be made in connection with sounding or boring tests. Usually, only an occasional test pit will be necessary, but test pits are invaluable in giving a complete understanding of the difficulty of foundation construction in an unknown locality. Usually one test pit to twelve or more borings or soundings will be all that is required.

The cost of test pits is very much greater than the cost of borings or soundings.

2. Rod Test.—There are many locations and conditions, where the sounding bar gives valuable and satisfactory information, although this may be negative in nature.

The sounding rod should be made from a solid bar of tool steel $\frac{5}{8}$ to $\frac{7}{8}$ in. in diameter, depending on the material to be penetrated. The bottom section should be pointed at one end and threaded for an outside coupling at the other. Where the earth is of such a nature that the bar can be churned down 6 or 8 ft. by hand, as is usually the case, the bottom rod should be from 10 to 12 ft. in length. This will eliminate one coupling and will decrease the friction for both driving and pulling of the rods. Additional sections 4 to 5 ft. in length, threaded both ends, should be provided in sufficient number to reach the desired depth. After being worked down as far as possible by hand, the rods are driven with a 10- to 12-lb. maul or hammer. The drive end of the rod should be provided with a special drive cap fitting over the rod and cushioned with hard wood to prevent injuring the threads. Fourteen-inch Stillson wrenches are used to add additional sections and in taking them apart.

Two or three men are required to make soundings, and depths of from 30 to 35 ft. are not unusual in this class of work. In soft ground 150 to 300 ft. per 8-hr. day may be sounded with such an outfit to depths of 20 to 25 ft.

It is impossible to determine, with any degree of accuracy, the nature of the material penetrated with a sounding rod, or of the hard stratum below, yet samples of the material can often be obtained by using a short piece of gas pipe on the bottom of the

rod instead of the pointed section. The nature of the hard stratum may often be judged to some extent by the action of the maul on the rod in driving. If rock has been encountered, there will be a sharp rebound to the hammer, and no further penetration. In sand, gravel or hard clay the blow will be more or less dead with slight additional penetration for some blows.

In case the rod brings up with a sharp rebound, indicating rock, other soundings should be made within a radius of a few feet to determine whether a boulder or the rock ledge has been encountered. If the soundings all show practically the same depth, bed rock is indicated, though knowledge of the locality and good judgment must be used in the interpretation of the sounding.

In pulling the sounding rod a short chain, lever, and block are used.

3. Auger Borings.—The earth auger is much used in investigating soil conditions, and, for soils where its use is adapted, gives, probably, the most accurate and reliable information of any of the various types of earth borings. Its field is limited, however, to such materials as will stay in the helix of the auger until brought to the surface, thus limiting its use to dry earth, clays, and other combinations in which there is sufficient clay to make the material cohesive. It is especially difficult to make auger borings in earth other than clays, where water cannot be kept out of the borehole. The earth auger is very often used to advantage in combination with other methods of making borings, especially in starting the boring where better speed can be made with the auger than with the chopping bit.

For some purposes the ordinary 6-in. post hole auger will suffice to secure the desired information. Although it is adapted to shallow holes only, holes to a considerable depth can often be drilled with such an auger by welding a short piece of gas pipe on the stem and adding additional sections of pipe. The post hole auger is especially adapted for examining the soil to a depth of from 4 to 6 ft. below the bottom of proposed footings, after the excavations for the footings have been made.

Earth auger outfits consist of a derrick and hoist (similar to the derrick and hoist described later for wash borings), auger bits adapted to the soils to be drilled, pipe or drill rods in sufficient quantity to reach the desired depths, drive pipe casing having a drive shoe at the bottom and a drive head at the top, a ram for

driving the casing, pulling block for pulling casing, and such small tools as may be required in the work.

In drill operations three to six men are used depending on the depth to be drilled. The derrick and hoist are used to raise and lower the auger and drill rods, in driving the casing or drive pipe, and sometimes in pulling the casing after it has begun to pull easily. The auger is turned down by the men with levers or wrenches attached to the rods. When the auger bit has been turned its length into the material being drilled, it is pulled out and cleaned, then lowered again, and the operation repeated. So long as the material is of such a nature that it will cling to the bit and be brought to the surface, good progress can be made by this method of drilling to a depth of 50 or 60 ft. Below this depth much time is required in taking out and replacing the rods.

If sand or gravel, or wet material that will not stand, is encountered, then the hole must be cased. This is done by removing the bit and drill rods and driving a casing of such diameter that the bit and rods will pass freely through it. On the bottom of the casing is a drive shoe having a beveled cutting edge to cut the earth as the casing is driven down. The casing or drive pipe should be made in sections about 5 ft. in length, and consists of ordinary pipe threaded at each end and connected together with ordinary outside couplings. After driving until a low penetration per blow is reached, the bit is put back, the casing cleaned out, and the bit worked ahead of the casing. The operation from this point on now consists of alternately drilling and driving the casing until the need of the casing is past or the boring completed. Better time will usually be made by drilling only a few feet ahead of the casing before pulling the rods and driving the casing.

When sand or gravel, or other material that will not be brought up by the auger, are encountered, the casing must be cleaned with a water jet, or sometimes a sand pump is used. The sand pump is an ordinary piece of pipe, small enough in diameter to go down the casing, and having at its bottom end a special valve. It is lowered into the hole by a rope and churned up and down until filled, when it is brought to the surface and emptied and the operation repeated. Often water must be poured into the hole to make the material "soupy" so that the sand pump will fill. Clean, heavy, compact sands cannot be removed with a sand pump, and when such material is encountered it must be

washed out with a water jet. For this purpose a line of ¾-in. pipe, open at the lower end, is lowered into the hole. The top of this pipe line is connected with a force pump, or with a water supply under pressure. The pipe then being lowered to the sand and the pressure turned on, the sand is brought to the surface by the force of the jet.

Where boulders are encountered, they must be broken up by using a chopping bit on the bottom of the drill rods, in the place of the auger bit, or by the use of dynamite, both of which processes are described under wash borings.

4. Wash Borings.—The wash boring method of making tests is now almost universally used for all deep and difficult foundations, as well as on much of the shallower work. Investigations by this method, or by this method in combination with other methods, often give unsatisfactory data, yet in the hands of experienced engineers, the information obtained by wash borings is, on the whole, reliable and satisfactory, to depths of from 150 to 200 ft.

In making test borings it is often necessary to start with a larger diameter casing and drill rods, going as far as possible with these, then telescoping a smaller diameter casing inside, and continuing the hole with the smaller casing and smaller rods.

4*a*. Equipment.—The equipment given below is such as may be used to a depth of 150 ft., the larger casing and rods going to a depth of 100 ft. and then continuing the work to 150 ft. in depth with the smaller casing and rods. The amounts of each size of casing and rods, of course, can be varied to meet the requirements of the particular locality and the depths to be drilled. Equipment may be secured from companies manufacturing well drilling or diamond drill outfits, or may be made up in any first class machine shop.

1⅞-in. drill rods, flush joints:
90 ft. with 10-ft. sections
10 ft. with 5-ft. sections

1⁵⁄₁₆-in. drill rods, flush joints
140 ft. with 10-ft. sections
10 ft. with 5-ft. sections

2½-in. casing, flush joints:
95 ft. with 5-ft. sections
3-ft. section
2-ft. section
1-ft. section

1½-in. casing, flush joints:
145 ft. with 5-ft. sections
3-ft. section
2-ft. section
1-ft. section

Bushings: 1 for 1⅞-in. rods to 2½-in. casing, and 1 for 1⁵⁄₁₆-in. rods to 1½-in. casing.
Cross chopping bits: 1 for 1⅞-in. rods and 1 for 1⁵⁄₁₆-in. rods.
Patent pipe tongs: 2 pairs of Brown's No. 3 and 2 pairs of Brown's No. 4.
Chain tongs: 2 pairs to take up to 2½-in. pipe.
Combination hoisting and water swivel: 1 for 1⅞-in. rods.
Hoisting plug: 1 for 1⅞-in. rods.
Special coupling: 1 for 1⅞-in. swivel to 1⁵⁄₁₆-in. rods.
Stillson wrenches: 2, 14-in. and 2, 24-in.
Coes monkey wrenches: 1, 10-in. and 1, 15-in.
Sister hooks to take 1¼-in. rope: 1 pair.
Sheave wheel with pivoted hook: 1, 10-in.
Hawser laid Manila rope: 50 ft. of 1⅛-in.
Pick, shovel, hand ax, 12-lb. hammer, hand saw, crowbar.
Pulling block with a set of wedges for pulling 2½-in. casing and another set for pulling 1½-in. casing.
Jack screws: 2 of heavy type.
Brace with bits: ¼-in. to ⅞-in.
Combination nippers and pliers: 1 pair.
50-ft. metallic tape: graduated to feet and tenths of a foot.
1-gal. oil can.
Hand oiler.
⅜-in. chain: 4 ft. long with hook and ring.
Three cornered and flat files, punch, cold chisel, screw driver, packing for swivel and pump, small amount of No. 10 and No. 14 soft wire.
Gas pipe to be used in clamping Brown's pipe tongs to the rods and to the casing; 3 pieces of 2-in. pipe, 3 in. long, lap weld, no threads; 3 pieces of 1½-in. pipe, 3 in. long, lap weld, no threads.
Brass lined, double acting force pump, about 4 × 4½-in. cylinder, with 10 ft. of 1½-in. suction hose and strainer, and discharge end fitted to ¾-in. water hose.
Best grade ¾-in. water hose, 30 ft. long, one end fitted to the ¾-in. connection on pump, the other to the water swivel.
25-hole electro-magnetic push battery.
Derrick consisting of:
  8-in. drum, 30 in. long, with 1½-in. square steel axle, and 2 straight handles, 5 ft. long.
  2, 10 × ¼-in. steel plates (round) for ends of the drum.
  Timber, straight grained and dense yellow pine:
    2 pcs., 3 × 4 in., 20 ft. long.
    1 pc., 4 × 4 in., 20 ft. long.
    1 pc., 4 × 4 in., 8 ft. long.
    2 pcs., 2 × 4 in., 11 ft. long.
  2, 1½-in. boxes for the drum.
  1 clevise, 8 in. × ⅞ in., with ⅞-in. pin, 14 in. long.
If necessary to take the derrick apart in making long moves, it should be put together with angles and bolts, and there will be needed:
  4, 4 × 4 × ¼-in. angles, 4 in. long, each leg punched for ⅝-in. bolts.
  4, ⅝-in. bolts, 3½ in. long.

12, 5/8-in. bolts, 5 1/2 in. long.
2, 5/8-in. bolts, 6 in. long.
2, 5/8-in. bolts, 7 in. long.

The drill rods should be made up of special heavy pipe, the 1 7/8-in. pipe weighing about 5 1/2 lb. to the foot, and the 1-in. pipe about 3.2 lb. per foot, made from 1 1/2-in. and 1-in. pipe respectively. The pipe or rods are threaded on the inside with square threads into which are screwed special couplings, about 6 in. long, of sufficiently heavy metal to make the joints as strong as the rods themselves. These joints are flush throughout their length, and should be very accurately fitted together so that a full bearing of the rod with the coupling is secured when they are screwed together. This is an important feature and should not be overlooked.

The casing should be made from lap welded steel pipe and, for the 2 1/2-in. casing, may be made from extra heavy pipe if the work to be done is of a difficult nature. Extra heavy pipe cannot be used for the 1 1/2-in. casing as there would not be clearance for the 1-in. rods. The 2 1/2-in. and the 1 1/2-in. casings are made from 2 1/2-in. and 1 1/2-in. pipe respectively.

The 1 1/2-in. casing must be flush jointed to telescope inside the 2 1/2-in. casing, and it is advisable, although not necessary, that the larger size also be flush jointed. If outside couplings are used, the friction of the earth against the pipe will be much greater and the casing will have to be driven. With outside couplings, the casing is also more difficult to pull.

Flush joint casing should have square threads, each piece, except the bottom one, having a male thread at one end and a female thread at the other. The bottom section should be plain pipe, square cut at the bottom, with a female connection at the top to protect the threads. These connections should be accurately milled at both top and bottom of the connection so that a full bearing will be obtained at both ends of the thread when the pieces are screwed together.

It will be found of great advantage if both the drill rods and the casing are cut to length, so that it will not be necessary to measure each piece as it is added in the drill operations. The combination lifting and water swivel is used in raising and lowering the rods, and allowing them to be worked back and forth or around and around in drilling, with the water passing through to wash up the choppings from the bit.

The cross chopping bits are made of hardened steel and have chisel shaped edges for chopping and breaking up the earth. Four holes near the bottom of the bit allow the water to escape and these holes are so drilled as to throw the force of the jet downward to the cutting edges.

An ample water supply under strong pressure is essential in the wash boring process. In city work where access to the city supply may be had, the hand force pump will usually not be necessary. In such cases sufficient water hose, or a pipe line,

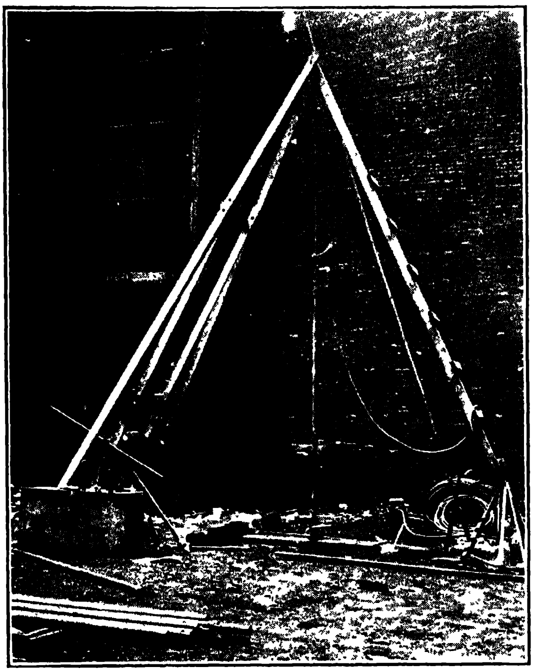


Fig. 1.—Derrick used in making wash borings.

will be required to reach the supply, in addition to the equipment enumerated. Where such supply does not exist and where there is no pond or stream in the vicinity of the boring, it will be necessary to haul the water, and this at times is a costly proposition.

The derrick must be strong and substantial, and should be so constructed that it can be quickly taken apart or put together, if subject to numerous long moves (see Fig. 1). If the work

requires many short moves it is advantageous to replace the straight handles on the axle through the drum with strong wheels of about 40 in. in diameter, one wheel rigidly fastened to the axle, the other loose. In moving with this arrangement, the single leg of the derrick is let down on the drum, the top end roped to the rear end of a wagon, and the derrick moved by team, on its own wheels.

Very often a "jack plank" is used in drilling. This consists of two 2 × 12-in. planks, one 16 ft. long and the other 12 ft. long, bolted together in the form of a T, the 16-ft. plank being the stem of the T. The two legs of the derrick stand upon the top of the T and the single leg on the far end of the stem. This "jack plank" gives a firm support for the derrick and also provides a support for the tongs in operating the casing.

Four to six or more men are required in drill operations by hand, depending on the depth of the hole and other difficulties attending the work.

The drill and pump may also be operated by power, a 3- to 5-hp. gasoline motor being the most satisfactory outfit for such purposes. The engine should be equipped with a hoist for raising and lowering the rods, and should have a special "nigger-head" on which the rope is operated, in churning the rods in drilling. Two men are required to operate a power outfit.

4*b*. Methods in Detail.—In starting a bore hole, it is of considerable advantage, where the ground will stand, to drill as far as possible either with a larger chopping bit, or with an earth augur, slightly larger than the casing to be inserted. This reduces the friction and is often of great help in difficult work. As soon as caving or running ground is encountered, the casing must be started. This is done by inserting the plain end of the bottom piece of 2½-in. casing in the hole that has been started. A pair of Brown's No. 4 pipe tongs are then clamped to the casing by means of one of the sections of 2-in. gas pipe, 3 in. long, driven over the handles of the tongs. The casing and tongs are then lowered until the tongs rest upon the "jack plank," or other support if a "jack plank" is not used. Another piece of casing is then screwed into the lower section, the tongs raised to the top of the added section, and the two sections lowered as before. This process is repeated until the casing reaches the depth drilled before the casing was started, or until it binds in the hole. If not down to the depth previously drilled,

the casing can usually be worked down still farther by turning on the tongs; when it cannot be advanced any farther by turning, the drill rods are inserted. In adding the sections of casing, should they become too heavy to handle by hand, the bushing is screwed into the top section and the casing then lowered by the derrick and hoist.

In starting the drill rods the cross chopping bit is screwed to the end on a drill rod. As the bit and rods are worked down, sections of drill rod are added until the bit has been forced some distance ahead of the casing; the rods are then raised until the bit is up in the casing, and the casing then turned down as before. The rods are supported, raised, and lowered by the rope attached to the bail of the water swivel and passing up over the sheave wheel and down to the drum of the derrick to which it is fastened. By means of the handles the men can hoist the rods with ease.

In drilling, the drum is made fast, with the bit resting on the earth, a few inches of slack being provided to allow the bit to advance into the earth. The rods and bit are then given a churning motion, by the men alternately pulling back on the rope and suddenly releasing it, causing the bit under the impact of the rods to hit the earth with considerable force, the drop usually being from 6 to 12 in. As the bit strikes, the foreman, who has hold of a pair of tongs clamped to the rods gives the rods a turn of about 90 deg., loosening up the earth into which the bit has sunk, which is then washed to the surface between the rods and the casing by the jet from the force pump. In some material, as sand or gravel, the rods cannot be churned down ahead of the casing on account of the sand filling in as fast as the rods are pulled out of it. In such cases the rods and casing must be worked down together, this being done by turning both rods and casing at the same time, the bit being at about the level of the bottom of the casing.

When the casing becomes tight, it can often be loosened by pulling it up a few feet and working it down again, repeating the operation several times if necessary. When the casing cannot be worked ahead any farther, the drill rods must be pulled out and the smaller casing and rods inserted, when drilling can be continued as before. If drilling in hard material that stands well when the larger casing stops, the larger rods should be worked ahead as far as possible before the smaller casing is

put in. The rods and casing being of the same diameter, the smaller casing can often be worked many feet ahead of the larger casing without the friction that would result if the smaller rods were inserted as soon as the larger casing becomes stuck. It may happen in some localities, that it will not be possible to reach the rock or the desired depth with two sets of casing; in such cases 3-in. casing may be used at the top, using a larger sized bit on the 1⅞-in. drill rods. Owing to the increased area of the space between the rods and the casing, the volume of water will have to be much increased, and if this cannot be accomplished by higher pressure by the pump, a larger pump will be required.

By the methods described above, the casing is always comparatively loose. When it becomes tight, a smaller diameter casing is telescoped inside and the work proceeds with smaller casing and rods; in this way the friction of the earth against the casing is eliminated for the depth of the larger casing.

Another method for making wash borings much used in some localities, is by driving the casing. Where the casing is to be driven, it is advisable to use ordinary pipe with outside couplings, as the flushjoint casing will not stand heavy driving. The casing is shod with a steel drive shoe with a beveled cutting edge. At the top is a steel drive head into which is screwed a hollow guide, the center of the drive head being drilled out to allow the drill rods to pass through. A heavy jar weight, about 300 lb., is raised by means of a rope over a sheave at the top of the derrick, and dropped over the hollow guide and striking the drive head forces the casing into the earth. Flush drill rods, such as already described, are used in drilling, the bit being pulled up in the casing while the casing is being driven. A short top piece of perforated casing must be used to allow the escape of the water and must be taken off of the casing in place and added to the new sections as they are put on. Two ropes, one operating the drill rods and the other the jar weight, are used in this method of drilling.

The casing can often be pulled by the use of the derrick and hoist alone; often a block and tackle are used; also the chain hoist is sometimes employed. The lever and chain, however, is probably used more often than anything else. When the casing becomes stuck and cannot be pulled by any of the above methods, casing clamps are fastened to the casing and jack screws used to start it. After it has been loosened a little, it can be pulled much more rapidly by one of the methods described above. A more

satisfactory appliance than the bolted casing clamps, is the pipe puller. This is a cast steel block with a hole through it considerably larger than the pipe to be pulled; this block is set over the casing to be pulled and wedge sections made to fit the casing are inserted. Jack screws are used under the pulling block to start the casing. When the jacks are loosened, a blow of a hammer causes the wedges to drop out and the block to become loosened.

In making investigations of the soil under rivers, lakes, and other bodies of water, a scow or catamaran may be used if the holes are comparatively shallow, and the casing can be pulled without the use of jack screws. In deeper and more difficult work under water, piles are driven, a working platform constructed, and drill operations carried on from this platform as on land.

When obstructions are encountered in drilling and they cannot be broken up and passed by the chopping bit, the hole is thoroughly washed out and a charge of 40 per cent dynamite lowered to the spot and exploded. If the obstruction is a small boulder, a single charge will usually suffice to clear the obstruction. If it is a large boulder several charges may have to be used before any impression is made upon it. If it is a very large boulder, or bed rock, any number of charges will have but little effect. In working through hard clay or hard pan, it will be found impossible to force the casing through it by the methods described above. In such cases, the drill rods are worked ahead of the casing a few feet, 6 to 8 ft. perhaps, and a string of dynamite used. This is made by connecting a number of sticks of dynamite end to end with small sticks of wood, 12 to 18 in. long, separating them. The exploder is usually inserted in the lower stick, the others being exploded by concussion. In using dynamite, the casing must always be pulled up several feet above the charge to be exploded, otherwise the casing will be damaged, perhaps so badly as to require the abandonment of the hole. For a single stick of dynamite, the casing should be raised 3 to 4 ft., while for the very heavy charges it may have to be raised 6 to 8 ft., or more.

To determine accurately the depth to and the thickness of the various strata encountered, and for the proper placing of dynamite, it is necessary to know at all times the exact amounts of drill rods and casing that are in the hole. A record should be kept and each piece of rod and casing entered as it is put on, and the totals extended so that the drill foreman knows at all times

the exact location of both rods and casing with respect to the surface. As stated before if the rods and casing are cut to length, the matter of measurements is much simplified.

4c. Records and Samples.—The interpretation of drill records and samples is a very difficult matter and requires experience and good judgment in this line of work. In augur borings, the earth is cut up, and in the case of hard clay especially, where the augur bit cuts into it very slowly, the sample obtained is not truly representative of the nature of the material in place. In wash borings the cuttings are caught in a bucket as they are washed to the surface, and allowed to settle; but, while such samples show the kind of earth passed through, they do not, usually, give much indication of the nature of the material in place as to its hardness and suitability for foundations.

The best method of taking samples, probably, is by means of a short section of pipe closed at the top and having vents at its top to allow the escape of water. This pipe should be beveled on the outside, at the bottom, to a sharp cutting edge so that the compression of the material being sampled will be reduced to a minimum. Samples taken in this way are usually satisfactory, and are truly representative of the material in place.

In sand and gravel formations, fairly reliable samples are washed up by the jet unless the individual grains are too large to pass between the rods and the casing. Generally, samples obtained in this way will contain more than the correct proportions of the finer grains where they vary widely in size. Samples of sand and gravel are also secured by means of the sand pump.

Samples, as soon as taken, and before they have a chance to dry out, should be placed in wide mouthed bottles and tightly corked or capped, to preserve the original moisture in the material. Gummed labels, on which are printed the location of the boring, and the depth or elevation of the stratum, should be stuck to the bottles for identification. In taking samples, every precaution should be taken to make them truly representative of the material sampled. If this is done, then the samples together with the record of the boring, will in most cases furnish information sufficiently reliable to enable the engineer to design the foundations for the structure in hand.

In field notes of bore holes, all matters and conditions pertinent to the work should be entered, such as the depth at which each stratum is encountered, the kind of material (whether sand,

clay, etc.), whether the material is hard or soft (readily determined by the action of the bit chopping into it), whether the casing is worked through it easily or the reverse, whether the material will stand without the casing, the location and nature of obstructions that may be encountered, where blasting had to be resorted to if at all, and any other items that may assist in the interpretation of the earth stratification.

4*d*. Cost of Borings.—The cost of borings in any location will depend on many factors, such as the depth and nature of the material, the location as regards transportation and the proximity to water, whether the work is bunched or scattered, amount of work to be done, and other conditions. Often the cost of organizing and moving to the work will cost much more than the actual drilling.

In 1897 to 1899 the United States Deep Water Ways Commission made nearly two thousand borings, averaging about 35 ft. in depth, through New York State and in Canada, at an average cost of 54 cents per linear foot. All types of materials were found in these borings. The work was scattered, but was of such a magnitude that efficient organizations could be put in the field to handle the work. At present prices for labor and materials, this price would probably be more than doubled. In Chicago, contract borings, 50 to 100 ft. in depth, cost (1921) $1.50 to $2.50 per linear foot.

5. Core Borings.—In verifying bed rock the core drill is generally used. It is also used to determine the nature of rock that will be encountered in driving tunnels for railroads, sewers, water supply, etc.

As the name implies, the core drill produces an exact core sample of the material penetrated, so that the information obtained by core drilling is accurate and reliable; unfortunately, its use is restricted to solidified formations, the core being ground up and washed away where the drill is used in the softer formations.

The essential equipment for core boring work consists of drill rods and casing, a rotary power plant (may be hand power for holes up to 500 ft. in depth), derrick and hoisting apparatus, force pump, and an automatic feed for forcing the drill bit into the rock.

Before core drilling operations can start, it is necessary to drive the stand pipe or casing down to rock, and to seat it in

the rock to effectually shut out all sand and earth that would tend to enter the bore hole from the surface of the rock, if casing were not driven. This often proves a difficult and costly operation, especially if it must be driven through sand, gravel, and boulders. This standpipe or casing is the same, in essential details, as that used and described for wash borings, and is driven to rock by one of the methods there described. There is one difference from wash borings, however, and that is that in core drill work the casing must be seated into the rock, whereas in wash borings, holes are often completed without driving the casing all the way to rock.

There are two principal types of core drills: (1) the diamond drill, using black diamond or carbon as an abrasive to cut the rock and (2) the shot drill, in which chilled shot is the abrasive agent. The essential features in the operation of one type is practically the same as in that of the other.

5*a*. Diamond Drill Method.—In making diamond drill borings, the drill rods carry a soft steel bit at their lower end, into which are set small pieces of black diamond or carbon, which, as the bit is rotated, cut an annular hole into the rock, leaving a center core undisturbed. Water forced through the drill rods by the force pump, washes the cuttings to the surface and keeps the bit cool. Just above the bit is a core barrel, usually about 10 ft. in length, and having at its bottom end a core lifting device which passes smoothly over the rock core as the rods are fed downward but automatically grips and breaks the core when the rods are raised. When the core barrel becomes filled, or oftener if desired or conditions require, the rods with core barrel are pulled to the surface and the core removed.

By keeping a careful record of the drilling process and of the core as it is removed, the location of the different rock strata, the thickness of the various strata, and the nature of the rock are known. Separating layers of clay or soft rock will be indicated by a loss of core and the amount of this loss will be indicated by the ratio of the core obtained, to the depth drilled to obtain it. Solid rock will usually give from 90 to 100 per cent of core.

Core samples, as they are taken from the core barrel, should be placed in order, in boxes especially made for the purpose, and properly marked for identification. For ordinary purposes in

foundation work the size of hole drilled will be about 1⅜ in., the core being 15/16 in. in diameter. In drilling this size hole, the smaller size rods described for wash borings are used.

Successful operation of the diamond drill requires that a skilled mechanic, with experience in this class of work, be in charge of drill operations at all times. While the operation of the drill mechanism itself is simple and requires no special knowledge over that required for other machinery and engines, the action of the bit in drilling varies greatly with different formations, and there is the possibility of serious loss through breakage of carbon in broken rock, also it is usually necessary to reset the bits at the job. The setting of a diamond bit calls for skill, practice, and a knowledge of the best arrangement of the diamonds for different formations, and to prevent loss by breakage or by the stones becoming loose and being lost.

Two kinds of diamonds are used in making diamond drill borings—namely, carbons and bortz. Carbons are opaque and non-crystalline, while bortz is semi-transparent and crystalline. Bortz, while very hard, will not stand pressure, due to its crystalline nature and is only used in the softer rocks. Carbon now costs from $90 to $100 per carat, the cost of bortz being about half that amount. Since a bit requires eight stones averaging 1 to 1½ carats each, it is seen that a diamond bit is a very costly affair. However, except in the case of breakage, the loss of carbon in drilling is not excessive, even in the case of the harder rocks.

On the U. S. Deep Water Ways Surveys already mentioned in connection with the cost of wash borings, 222 ft. of very hard quartzite, 444 ft. of hard limestone, 363 ft. of hard sandstone, and 880 ft. of shale, were drilled with an average loss of carbon of one carat to 76.5 ft. drilled. In the quartzite the loss was one carat to 57 ft. drilled, while in one case 83 ft. of limestone and 310 ft. of shale were drilled with a loss of but little over ⅝ carat of carbon. On this work there was a great deal of broken rock formation and the loss of carbon by breakage was about 35 per cent of the total loss. In the quartzite an average of about 25 ft. was drilled before the bit had to be reset; in the softer stone the amount drilled per bit was from 22 to 392 ft. The average for the job was 87 ft.

On this work the average number of feet drilled per day, both earth and rock, for the actual days on the work, was 13.1 ft.,

and for the actual time on drill operations, not counting delays in moving, loss from storms etc., the average per day was 16.2 ft. The rate of sinking standpipe, actual time working, was 1.7 ft. per hr.; and the rate of drilling in rock, actual hours drilling, was 2.53 ft. per hr.

The distribution of average cost per foot on this work was as follows:

Rental of drill outfit	$0.896
Carbon	0.372
Repairs	0.062
Labor	0.714
Teaming	0.355
Superintendence	0.390
Fuel for boiler	0.050
Lumber	0.023
Core boxes	0.014
Freight and express	0.135
Traveling expenses	0.093
Sundries	0.033
Total	$3.137

These costs are on the basis of 1898 prices; present prices would be much higher. In explanation of the cost of this work it may be added that the work was started in Oct. 1898 and finished in May 1899, that the intervening winter was a severe one, and that the frequent moves necessary, were many of them made over very bad roads. Thirty-three moves were made by teams, covering about 110 miles. Five moves were made by rail, covering nearly 500 miles and these moves were also costly in delays and time lost.

5b. Shot Drill Method.—Core borings by the shot method requires very similar equipment to that used in diamond drill work except that a shot bit instead of the diamond bit is used. The bit is made of special carbon steel, of heavier wall than the core barrel. A slot is cut at the bottom of the bit to allow the circulation of water. Chilled steel shot are used to cut the rock and these are fed as needed to the bit through the drill rods, by a special valve at the water swivel. In the place of a special device for lifting the core, a handful of coarse sand or fine gravel is forced down through the drill rods and this acts to wedge in between the core and the sides of the core barrel and

locks the core so that it can be brought to the surface. In shot borings the minimum size of hole is about 2½ in.

This method of drilling has the disadvantage in that inclined holes, making an angle of more than about 70 deg. with the vertical, cannot be drilled, because of the shot gathering on the low side of the hole as the bit is fed into the rock. Speed of drilling is also slower than with diamond drilling, and in very hard rock, it is difficult to make headway at all. The initial cost of shot core drills is much less than with diamond drills. On the other hand, diamond drill cores of a diameter greater than 2 or 3 in. are not often made, while cores of from 12 to 16 in. are not unusual by the shot method, and cores up to 30 in. in diameter may be taken if required.

6. Load Tests.—Load tests are made to determine the safe carrying capacity of soil, rock, piles, etc. Load tests on rock are usually confined to the softer and broken rocks, shales, etc.; in the case of solid, hard rock, its capacity is known to be more than the loads that it is proposed that it shall carry, so that testing it is not necessary.

There is at present, no standardization of practice in making load tests. Most of the large cities require that such tests be made where uncertainty exists as to the safe bearing power of the soil, but the method of making the tests and the interpretation of the results are in each case left entirely to the commissioner or superintendent of buildings.

In Chicago, load tests are required to be made on concrete piles and for these tests the ordinance specifies that "one-half of the test load shall be allowed for the carrying load, if the test load shows no settlement for 24 hr., and the total settlement has not exceeded one one-hundredth of an inch multiplied by the test load in tons." The Chicago Ordinance does not specifically call for test loads on soil, though they are required by the city authorities to be made where question arises as to the carrying capacity of the soil at a proposed building site.

The New York City building ordinance reads, "when doubt arises as to the safe sustaining power of the soil upon which a building is to be erected, the Superintendent of Buildings may order borings to be made, or he may direct the sustaining power of the soil be tested in accordance with the methods established by the rules of the Superintendent of Buildings. . . . . these instructions being in part as follows:

The soil shall be tested in one or more places as shall be determined or the conditions warrant, at the level at which the proposed footings are to be placed.

Each test shall be so made as to load the soil over an area of not less than 4 sq. ft. in any one place.

The accepted safe load shall not exceed two-thirds of the final test load.

The loading of the soil shall proceed as follows:

(a) The loads per square foot which it is proposed to impose on the soil shall first be applied and allowed to stand for at least 48 hr. undisturbed, measurements or readings being taken each 24 hr. or oftener in order to determine the settlement if any.

(b) After the expiration of 48 hr., the additional 50 per cent excess load shall be applied and the total load allowed to remain undisturbed for a period of at least 6 days, careful measurements and readings being taken once each 24 hr., or oftener, to determine the settlement.

The test shall not be considered satisfactory or the result acceptable, unless the proposed safe load shows no appreciable settlement for at least 2 days, and the total test load shows no settlement for at least 4 days.

It will be noticed that these rules take care of any slight initial settlement that may occur when the load is first applied, it being very difficult to secure a full bearing of the post or plate on the soil until some of the load has been applied. The rules also provide for the natural yielding of the soil under the load, but settlement must not continue, the excess load test showing no settlement for at least 4 days.

The area loaded, in making load tests, varies from 1 sq. ft. to areas approaching and occasionally equal to the area of the proposed footings. In general, the larger the area loaded, the more representative the test will be of the sustaining power of the soil.

In loading tests on 1 sq. ft. area, a single post, usually 12 by 12 in., is set on end on the soil to be tested, or upon a cast-iron plate bearing on the soil. On top of this post is constructed a platform on which the testing weights are balanced. Often four such posts are used, on which a platform or loading bin is constructed. Settlements are determined by making observations with a Wye level and level rod, on a pin or bolt set in the top of the post under the load. Sand, cement, brick, pig iron, steel rails, or other materials convenient to the test are used for loading the platform. In placing the load, care should be taken to cause as little vibration of the load as possible, since vibration, transmitted through the post to the soil under test, will cause additional settlement.

The hydraulic jack is occasionally used in making load tests. Where used, a reaction for the top of the jack must be provided. For this reaction a load may be built up on blocking or cribbing, centered over the area to be tested. This load should be slightly in excess of the test load, and should be balanced on timber or steel beams under which there is a cross girder to take the reaction of the jack. In using the hydraulic jack for making load tests, slight leakages of oil from around the piston or plunger, and settlement of the plate under load, will cause a drop in the pressure of the jack, so that careful supervision is necessary in making such tests to keep the pressure pumped up to the test requirements so that it will remain constant.

In making load tests on concrete piles, if the concrete pile has been poured in place, it is necessary to allow ample time for the concrete to set; usually 21 to 28 days will suffice. The loading platform is balanced directly on the pile and the loading carried on in the same way as described for tests on the soil. It has been determined that the supporting power of a pile increases with the length of time that the pile has been driven, and that a pile slowly loaded will carry more than one where the load is applied in a very short period of time.

BEARING POWER OF SOILS AS INDICATED BY BUILDING CODES[1]

By Tirrell J. Ferrenz

7. Reliability of Codes.—The bearing power of foundation beds as indicated by the building codes of our principal cities may be considered in general as representative of good standard practice. An inspection of the table on pp. 24 to 27 shows a reasonably close agreement in the bearing values assigned to the various classes of soils, but there is, however, a marked divergence in the terms used to describe the same soil condition. This is especially noticeable with reference to the meaning of the word "loam" as used in different sections of the country. The lack of standardization in other terms is also apparent.

These faults are no doubt due in a large measure to the fact that many codes are either a reflection of the personal experience of one or two men or else are copied from other cities and then added to or altered in an attempt to make them fit local condi-

[1] For characteristics of soils, etc., see Appendix *A*.

tions. In contrast to this condition, quite a few codes have been developed bit by bit through the labor of large committees or boards of technical experts and bear the weight of competent authority. The standard building code prepared by the National Board of Fire Underwriters is an excellent example of the latter type. This code was not designed for any particular set of local conditions but is intended for general usage as a thoroughly practical non-political code.

8. Special Provisions.—Many codes contain special provisions or requirements which are of considerable importance. Some of the more note-worthy of these are as follows:

(*a*) Foundations must not rest on filled ground or loam (Chicago) or on any soil containing a mixture of organic matter (Chicago, Phila.), but shall be extended to firm undisturbed natural soil (Chicago, Los Angeles).

Loam or soil containing organic matter shall not be used to support any building over one-story in height (Seattle, Portland) and no building over 15 ft. in height shall be built on filled ground unless "special precautions" are taken (Cleveland).

Tests must be made on filled ground (Bridgeport) and on "inferior soils" (Dayton).

(*b*) Materials shall be in relatively thick beds if full bearing value is to be used; otherwise, if underlaid by a softer material, the value assigned to such material shall be used (Boston). Soils shall be of sufficient thickness to distribute the load over the requisite area of underlying soil (Seattle).

(*c*) In case a structure is to rest partly on solid rock and partly on yielding soil, the bearing capacity of the yielding soil shall be taken at not more than one-half of the value of that particular soil ordinarily allowed (New York, Boston).

(*d*) Where foundations are to rest on bed rock, the rock must be leveled or benched to receive the foundations (Buffalo, Baltimore, Pittsburgh).

9. Use of Codes.—In deciding upon the allowable bearing power of soil to be employed in any particular case, the provisions of the local building code should not be accepted blindly, inasmuch as no set of rules can be promulgated which will cover all the vagaries of soil conditions that may be encountered in any locality. Records of excavations on adjoining lots, or other property in the vicinity, are helpful in checking the engineer's judgment but should not be relied on exclusively. Observations

BEARING POW-

	New York	Chicago	Philadelphia	Detroit	Cleveland	St. Louis	Boston	Baltimore	Los Angeles	San Francisco	Buffalo	Milwaukee	Washington	Newark	Cincinnati	Minneapolis	Seattle
Alluvial soil					½							½					
undisturbed																	
Clay, dry																	
dry, thick beds															4		
firm	2																
firm, dry								3		3		3	3	3		3	2½
firm, wet																	
good, solid, natural						2½											
hard				4	3			4	2–4	4	3½		4	4		4	3½
hard, dry	4		3½														
hard, blue							5										
hard, yellow							6										
medium stiff							4										
moderately dry															2		
moderately dry, thick beds																	
ordinary																	
pure		1½															
pure, dry, compressed		2¼															
soft	1							1	1	1		1	1	1		1	1
soft, confined							2										
soft, wet																	
wet					1												
Clay and sand, dry					2												
dry and firm								3									
in layers	2																
in layers, wet																	
mixed	2	1½								2							
moderately dry																	
ordinary, in layers								2									2
ordinary { in layers				2								2	2	2		2	
ordinary { wet and springy																	
wet					1½												
Gravel	6						6				3½	5			5	4	
coarse								6		6							
deep excavations																	5
firm									2–4								
loose			3½														
stiff				4									4	4			3½
thick beds					8												
well-cemented			6														
well-compacted																	
Hard pan	10						10	12				6					
gravel, deep excavations																	8
Loam										3							
firm, dry								3					3	3		3	
sandy									1								
Quicksand					½							½					
Rock																	
bed												20					
fairly hard																	
hard	40							20		20							
hard, under caissons								24									
ledge, disintegrated							5										
ledge, solid							100										
medium	15																
soft	8																
solid					10												
very hard, native bed																	

ER OF SOILS

Indianapolis	Jersey City	Rochester	Portland	Denver	Providence	Columbus	Louisville	Oakland	Atlanta	Birmingham	Syracuse	New Haven	Memphis	Dayton	Bridgeport	Grand Rapids	Albany	Fort Worth	Sprngfield, Mass.	Spokane	Wisconsin State Building Code	Nat'l Board Fire Underwriters
					½–1	½							½								½	
															1							
																3						
					3–5	4								4								
	3	3					2½	3	2–3	2–3	3	3					3		2	3	3	3
												2										2
3																						
	4	6					4	4	3–4	3–4		4				4	4	4	4	4		4
			4										3		4							
			3										2		2							
					2–3	2								2								
			2	2							2					2						
	1	1	1		1	1		1	1	1	1	1			1	1	1	1	1	1	1	1
													1	3¼								
																2						
												2										2
								2			2									2		
			3																			
	2	2							2	2	2						2	2	1½		2	
				3							4	4		5								4
				5				6												8		
4	4	6				4			3–4	3–4							4	4	4			
			8		8–10	8							8									
															6						5	
		10										8–10			6		4				6	8–15
				1				3												3		
	3	3					2½		2–3	2–3	3						3	3				
					½–1	½							½								½	
			16			5–25																15–72
															6							
								20							20					200		
					25–50																	

BEARING POWER OF

	New York	Chicago	Philadelphia	Detroit	Cleveland	St. Louis	Boston	Baltimore	Los Angeles	San Francisco	Buffalo	Milwaukee	Washington	Newark	Cincinnati	Minneapolis	Seattle
Sand			3½														
clean, dry																	
coarse	4							4									
coarse, dry or wet							5										
coarse, firm																	
coarse, very firm				4									4	4		4	3½
coarse, thick beds					8												
coarse, well-cemented																	
coarse, well-packed												5			4		
compact							6			4							
compact, well-cemented												4					
dry																	
fine																	
fine, dry	3						4			3					2		
fine, dry, thick beds					4												
fine, clean, dry																	
fine, firm, dry				3				3				3	3	3		3	2½
fine, wet					2												
fine, wet, confined							3										
firm, pure		2½															
firm, deep excavations																	5
moderately compact																	
ordinary																	
soft																	
wet	2																
Sand and gravel																	
compact															5		
compact, well-cemented															8		
firm, coarse																	
Sandstone																	
Shale							10			10		6					
hard								12									
hard, below frost					6												
hard, under caissons								18									
Stone and clay, stratified															4		

of the amount of water present and the compactness of the soil are also valuable. Wherever the work is of importance or where there is any doubt about the proper bearing value to use, suitable tests or borings should be made on the site, to determine the actual conditions.

10. Table of Bearing Values.—The table on pp. 22 to 25 gives the bearing power of soils in tons per square foot as pre-

SOILS.—*Continued*

Indianapolis	Jersey City	Rochester	Portland	Denver	Providence	Columbus	Louisville	Oakland	Atlanta	Birmingham	Syracuse	New Haven	Memphis	Dayton	Bridgeport	Grand Rapids	Albany	Fort Worth	Springfield, Mass.	Spokane	Wisconsin State Building Code	Nat'l Board Fire Underwriters
				2							2											
					2–4	2						2										
				5							4	4										4
4			4													4						
	4	6					4		3–4	3–4							4	4	4			
			8		8–10	8							8									
														4	6						5	
								4												4		
					4–6	4						4									4	
																3				3		
															4							
								3														
														2								
	3	3					2½		2–3	2–3	3	3					3	3	2		3	3
												2										2
															2							
			2																			
															1							
											4											
														5								
																6						
												15										
								10												10	6	
					10–15																	
														4								

scribed by the building codes of 38 leading cities, together with one state code and the standard building code recommended by the National Board of Fire Underwriters. The cities are listed in the approximate order of their population. An attempt has been made to quote the exact wording used in the codes to describe the various soil conditions but this has not been possible in all cases owing to the limited space.

SECTION 2

EXCAVATION

By A. B. McDaniel

1. General Considerations.—Excavation is one of the most neglected but most important elements in all classes of construction work. In the consideration of this subject, it is important to keep in mind that the most efficient tool or machine must be utilized if the required results are to be secured in the least time, and at a minimum of cost.

The best method and tool or machine to use in any particular case depends on many factors, some of which are variable and uncertain. Some of these factors are: Magnitude of work, area over which work extends, nature of material to be handled, length of haul, cost and availability of fuel, supplies and labor, location of work with relation to transportation facilities, climate, etc.

When the job is small, inexpensive equipment should be used to keep down this element of overhead cost. However, on work of large magnitude, the distributed cost of expensive machinery would only slightly affect the final unit cost of the work.

Light, soft soils do not require any preliminary loosening and can be handled directly by any form of hand or power tool. Dense, tough and hard soils must first be loosened by plowing or blasting, and generally the size and weight of the loose material is such as to require power machinery for its disposition.

The wasting or spoiling of the excavated material generally requires some hauling device or means of transportation which must be considered in the planning of the work.

The location of the job with respect to transportation facilities, availability of fuel, labor, and supplies, and the machinery available, are important governing factors, which in some cases may closely limit the method and tool to be used.

2. Shallow Excavation.—This form of excavation generally occurs in the basement excavation for small buildings, the

foundation excavation for buildings without basements, for retaining walls, low dams, elevator pits, and similar structures. If the area covered is small, hand tools may be used to best advantage, but if a large space is to be excavated, scrapers, graders, or some form of small, portable power excavator should be used. For example, the excavation of the foundation pit for a retaining wall of small extent, or bridge abutment, could be economically done by hand shoveling, while the excavation of the basement for a factory building should be made with scrapers or a power shovel.

2a. Tools for Loosening.—Generally the surface soil for shallow excavation can be removed directly, without preliminary loosening, but the denser, tougher sub-surface soils—such as indurated gravel, hardpan, blue clay, etc.,—must be loosened before removal, especially if power machinery is not to be employed. The method and tool to be used will depend largely on the size of the job, the nature of the soil, depth of cut, etc. The more common tools for loosening are the mattock, the pick, and the plow.

The mattock is a long-handled tool, shaped like a pick-axe, but with blades set at right angles to each other. This tool is ordinarily used for grubbing, cleaving and trimming an area preparatory to loosening.

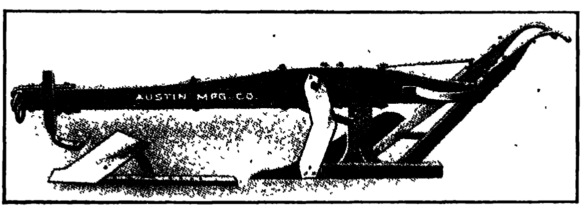


FIG. 1.—Typical railroad plow.

The pick is universally used for hand excavation in restricted areas, such as foundation trenches, pits, corners, and in places inaccessible to a power excavator. This tool has either two points, or one end pointed and the other wedge shaped. The amount of material that can be loosened with a pick during a 10-hr. day depends on the laborer, the depth of cut, the material, working conditions, supervision, etc., but will average about

8 cu. yd. for hardpan, 12 cu. yd. for gravel, and 18 cu. yd. for dense clay.

The plow is the best type of tool to use in the loosening of dense soils where there is unrestricted space. The "railroad" or "pavement" type of plow should be used wherever practicable, as its heavy, wedge-shaped share especially adapts it for this class of work (see Fig. 1). A two-horse team, driver, and man to hold the plow will ordinarily loosen about 400 cu. yd. of average soil per 10-hr. day. A four-horse team, and three men, will loosen about 200 cu. yd. of dense, tough soil in a 10-hr. day.

Table 1 gives a suggestive idea of the cost of plowing for various kinds of soil conditions:

TABLE 1.—COST OF PLOWING[1]

(Wages $1.50 and horse-keep $1.00 per 10-hr. day)

Soil	Labor	Cu. yd. per hour	Labor cost per cu. yd. (cents)
Loam	1 driver, 1 holder, 2 horses	50	1.0
Gravel and loam	1 driver, 1 holder, 2 horses	35	1.4
Fairly tough clay	1 driver, 1 holder, 2 horses	25	2.0
Very hard soil	1 driver, 1 holder, 4–6 horses and 2 men on plow beam of rooter plow	15–20	5–8
Ordinary soil	1 driver, 6 horses, on gang plow	40	1.9

Blasting has been successfully used in recent years for the excavation of trenches and pits, and for the loosening of the soil to facilitate its excavation. Sub-soil blasting can also be used to advantage in the breaking-up of hardpan and indurated gravel in the excavation of pits and trenches. Open trench blasting is done by charging holes, spaced 24 to 32 in. apart for small channels and 48 to 52 in. apart for large channels, with one-half or one cartridge of Forcite or other specially prepared dynamite.

[1] From "Earth Excavation" by GILLETTE, "Mining Engineers' Handbook," PEELE.

Sub-soil blasting is accomplished by loading holes 3 to 4 ft. in depth and from 5 to 18 ft. apart, with one-half to one cartridge of powder or dynamite.

In 1914, the writer used sub-soil blasting for the loosening of the soil for the excavation of footing pits for a reinforced concrete factory building. The pits were from 12 to 15 ft. square and about 7 ft. deep. Each pit was blasted by driving two holes, 1½ in. in diameter, 6 to 8 ft. on centers, and about 4 ft. deep, in each of which was placed a cartridge of 40 per cent dynamite. Each set of holes was exploded simultaneously by an electrical firing machine. The soil consisted of ashes and cinders to a depth of about 2½ ft., then a 1-ft. layer of peat resting on 3 ft. of red clay, which was underlaid with a dense shaly gravel. The blasting loosened the soil to such an extent that a crowbar was easily thrust down its full length into the soil.

2b. Hand Tools.—The principal hand tool used in excavation is the shovel. This tool is made with a long or short handle, and with round, square or pointed blade. The short D-handle shovel provided with a round-ended blade is the best form to use in the removal of dense, stiff soils. For the handling of soft, loose soils such as sand and gravel (especially if handled on or off boards), the short-handled, square-ended shovel is the most serviceable. The long-handled, round-ended shovel is the best form for the removal of ordinary soils where the material must be lifted.

The amount of material that can be handled during a 10-hr. day depends on the character and condition of the soil, the location of the material (whether in pit or bank), the method of work (whether continuous, spasmodic or systematic), the method of disposal, the kind of shovel used, and the efficiency of the laborer. On an average, a laborer can shovel loose material and elevate it upon a platform or into a wagon at the rate of from 15 to 10 cu. yd. per 10-hr. day, for lifts of from 3 to 5 ft., respectively. These quantities should be reduced to from 8 to 5 cu. yd. for a dense, tough clay or hard gravel.

Table 2 gives a compilation of data as to the cost of shoveling with various kinds of soil conditions:

TABLE 2.—COST OF LOADING BY SHOVELING[1]

Method	Cu. yd. per man-hour	Cost per cu. yd. Wages 15 cts. per hour (cents)	Authority
Mud into wheelbarrows	0.8	19.0	M. Ancelin
Gravel into wheelbarrows	1.7-2.7	7.0	M. Ancelin
Earth into wheelbarrows	1.6-4.8	5.0	M. Ancelin
Earth into wheelbarrows, average	2.2	7.0	M. Ancelin
Earth (all kinds) into wagons	2.1	7.5	Cole (a)
Earth into wheelbarrows	2.8	5.25	Gillespie
Earth (all kinds) into wagons	2.0	7.5	D. K. Clark
Sand into cars from high face	1.8	8.25	Gillette (b)
Plowed gravelly soil into wagons	1.3	11.3	Gillette (c)
Iowa soil	1.5-2.0	8.5	J. M. Brown
Iowa soil	2.8	5.4 (d)	J. M. Brown
Clay and gravel into carts	1.0	15.0	E. Morris
Loam into carts	1.2	12.5	E. Morris
Sandy earth into carts	1.4	10.75	E. Morris
Loose sand into carts	2.0	7.5	G. A Parker
Clay, tenaceous, Chicago	1.25	12.0 (e)	G. A. Parker
Hardpan into low dump cars	1.5	10.0	Gillette
Average earth	1.75	8.6	Gillette

(a) 10 miles, Erie Canal. (b) 10,000 cu. yd. bank measurement. (c) 20,000 cu. yd. in embankment. (d) A rush job. (e) Spaded out and handled with forks.

The following example of basement and foundation excavation is suggestive of the methods and output in this class of work:[2] In the excavation of a cellar, 14 shovelers loaded 23 wagons of 1½ cu. yd. (loose measure) capacity in 75 min., or at the rate of one wagon load per shoveler in 45 min. Each shoveler handled about 20 cu. yd. (loose measure) or 16 cu. yd. (bank measure) in a 10-hr. day. Later, 8 shovelers loaded the 23 wagons in from 3 to 5 min., for each wagon load, the average of 10 loads being 4 min., or at the rate of 27 cu. yd. (loose measure) or 21 cu. yd. (bank measure) per man per 10-hr. day. The length of the haul was 4350 ft. over level pavements, except at the pit and the dump, and the average round trip time was 29 min. The earth was loosened by a single team plow with driver and plow holder, at the rate of 300 cu. yd. per 10-hr. day. A unique method of excavating foundation pits is described in Art. 2c, p. 37.

2c. Scrapers.—The scraper consists of a steel pan with a cutting edge, and rests directly on the ground or is mounted on a two-wheel or four-wheel frame. The scraper is

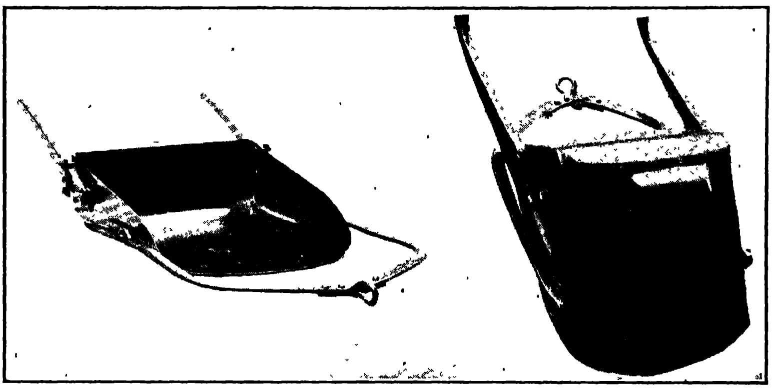
[1] From "Earth Excavation" by GILLETTE, "Mining Engineers' Handbook," PEELE.

[2] From "Earthwork and Its Cost," H. P. GILLETTE.

propelled by a two-horse team, and in hard or dense soil a snatch-team or traction engine is used to aid in loading.

The types of scrapers in general use are: (1) The drag or slip scraper, (2) the Fresno scraper, (3) the two-wheel scraper, and (4) the four-wheel scraper.

Drag Scraper.—The drag scraper consists of a steel scoop provided with a bail for the attachment of a team and two handles for its guidance (see Fig. 2). This form of scraper varies in size from 3½- to 7-cu. ft. capacity, but, in estimating output, allowance should be made for the fact that soil in scraper is loose and rarely fills the scoop.



(Courtesy of Western Wheeled Scraper Co.)

Fig. 2.—Front and rear view of drag scraper.

Drag scrapers are efficient up to hauls of 100 ft. and can be used advantageously up to 200-ft. hauls. A two-horse team and scraper can move in a 10-hr. working day, the following average amount of loose material:

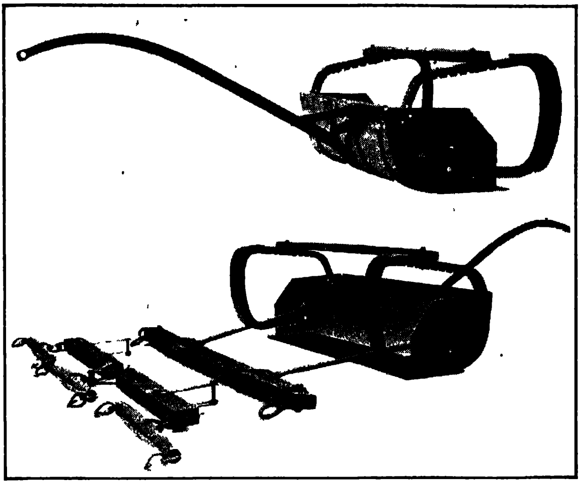
Length of haul (ft.).........	25	50	100	150	200
Output per day (cu. yd.)....	70	60	50	40	35

The scrapers should be used in groups of from 3 to 12, depending on the material, size of job, and length of haul. The driver generally loads the scoop, which can be filled directly in soft and loose soil but requires preliminary loosening in hard and dense soils.

Fresno Scraper.—The Fresno or Buck scraper has a long, narrow pan which rests directly on the ground. Fig. 3 shows two positions of this type of scraper. The capacity of the

scraper varies from 12 to 18 cu. ft. and the length of cutting edge from 3½ to 5 ft.

The economical haul of a Fresno scraper is 300 ft. and under average working conditions it will move from 60 to 125 cu. yd. of sandy clay soil, with a haul of from 75 to 150 ft., in a 10-hr. day.



(Courtesy of Western Wheeled Scraper Co.)

Fig. 3.—Rear and front view of Fresno or Buck scraper.

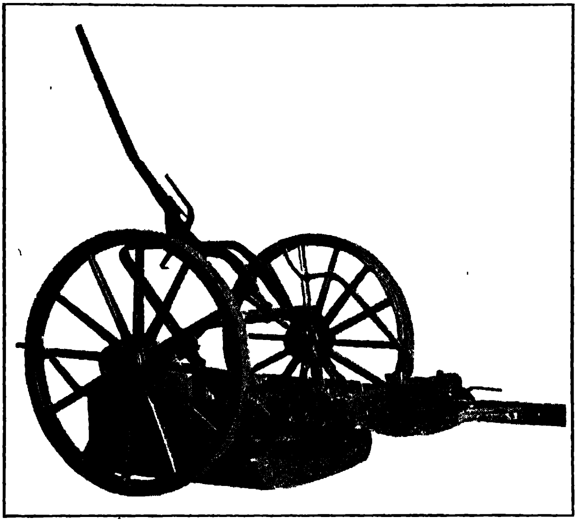
Two-wheel Scraper.—The two-wheel scraper consists of a steel box mounted on a single pair of wheels, and provided with levers for the raising and lowering and dumping of the pan, while the scraper is in motion. An automatic end-gate is sometimes attached to the front of the pan to prevent loss of material, especially when operating on steep slopes. Fig. 4 shows a scraper in the loading position. The capacities for two-wheel scrapers vary from 9 to 16 cu. ft.

The two-wheel scraper is an efficient earth mover for hauls of from 200 to 800 ft. A two-horse team and scraper can move in a 10-hr. working day the following average amounts of loose material:

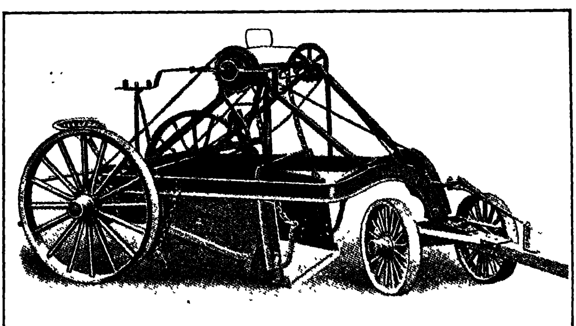
Length of haul (ft.)	100	200	300	400
Output per day (cu. yd.)	50	50	40	30

The scrapers should work in groups of from 4 to 6 for hauls up to 400 ft., and in groups of from 8 to 12 for longer hauls. One

man, in addition to the driver, is necessary to load and dump the scraper, and in hard tough soils, two men may be required for loading the larger size machines. As in the case of drag scrapers,



(*Courtesy of Western Wheeled Scraper Co.*)
FIG. 4.—Two-wheeled scraper in loading position.



(*Courtesy of Baker Mfg. Co.*)
FIG. 5.—Four-wheeled scraper.

the wheeler rarely leaves the excavation filled to its rated capacity. For long hauls and in dense, hard material, it is desirable to use shovelers to heap up the pans to complete the loading operation.

Four-wheel Scraper.—The four-wheel scraper comprises a pan having a rated capacity of ½ or 1 cu. yd. suspended on a steel frame, which is supported on two trucks. As is shown in Fig. 5, the pan is hung so that the cutting edge just touches the ground surface in its loading position. The pan is operated by four levers, which are within the reach of the driver who is seated just behind and on the right hand side of the rear truck. The motive power is a team of horses, and a snatch team or traction engine is used in loading.

The four-wheel scraper is about 100 per cent more efficient than the two-wheel scraper for 200-ft. hauls, and this efficiency increases with the length of haul up to about 2000 ft. The output of a 1-yd. scraper during a 10-hr. day, will vary from 400 cu. yd. for 3 scrapers with a 400-ft. haul to the same amount for 10 scrapers operating on a 1300-ft. haul. The unit cost of excavation for the first case will be one-half of that in the second case.

Field of Use.—The scraper is an efficient form of excavator for shallow cuts over large areas, such as cellars or buildings, foundation pits for the footings of walls and other structures, reservoirs, basins, etc. The type of scraper to use in any particular case depends on the nature and condition of the soil, length of haul, and size of job. For work of small magnitude and haul of 200 ft. or less, the slip or buck scraper could be used economically, but for longer hauls, with tough, dense soil the wheel scraper should be used. In cellar, pit, or trench excavation, where the cut varies from 1 to 4 ft. and the haul is from 500 to 1000 ft., 7 to 10 scrapers, loaded by a traction engine, can remove 500 to 800 cu. yd. of clay and loam in a 10-hr. working day.

Wheel scrapers and hand excavation were used in the construction of a seven-story reinforced-concrete factory building for the Stanley Works at New Britain, Conn. in 1914. The building is 63 ft. wide, 203 ft. long, and has no basement. The building site was a storage yard surrounded by one-story frame structures. Test pits revealed the nature of the soil as ashes and cinders to a depth of 2½ ft. below the surface. Underlying this fill was a 1 ft. strata of peat, then 3 ft. of red clay, and then the foundation material of a dense shaly gravel. In many places glacial boulders of from 1 to 4 ft. in size were encountered. At an average depth of 7 ft. below the basement sub-grade (approximately average ground level), water was found, and in several pits flowed in a steady stream, requiring the use of diaphragm or small capacity centrifugal pumps.

The problem was peculiar and difficult in this case, since there was to be no basement and the net excavation was the removal of sufficient material

for the building of 18 interior footings 15 ft. square and 6 ft. 9 in. below subgrade, twenty-two exterior footings 12 ft. square, and four corner footings 10 ft. square and 7 ft. 3 in. below subgrade. On account of elevator pits and a conduit tunnel-terminal, two interior column footings were carried to a depth of 10 ft. 6 in. and one interior and two exterior footings to a depth of 8 ft. 6 in. below subgrade. Allowing for side walls, column stubs, and tunnel, the total gross excavation required was about 2500 cu. yd., with a back fill to subgrade elevation of about 1500 cu. yd. It is, therefore, evident that 1000 cu. yd. of material was to be permanently removed and wasted. After a preliminary study of the conditions it was decided to scrape off the 1000 cu. yd. of superfluous material from the surface with two-wheel scrapers, and then excavate the footing pits to grade with pick and shovel.

The cinder fill was found to be very dense and compact, and a two-horse plow was necessary to loosen up this material for the scrapers. Four, five and six two-wheel scrapers were used on consecutive days to remove this surface material from the building site to the dump, which was located about 300 ft. from the south end of the site. Hence, the average length of haul was about 400 ft. Each scraper was operated by two horses and a driver, and no snatch team was used for loading, as the plowing up of the material rendered it loose enough to be loaded easily into the scrapers by the regular team alone. The net capacity of each scraper was ½ cu. yd. The average time for the round trip for the average haul was 4½ min. Two men were used to load the scrapers and two to dump and spread the material. A foreman supervised the loading, and a clerk at the dump kept a time record and superintended the dumping. Following is a schedule of the labor cost of the wheel-scraper work per 9-hr. day:

1 foreman at $6 per day	$ 6.00
4 laborers at $0.25 per hour	9.00
1 clerk at $10.00 per week	1.67
6 teams at $0.60 per hour	32.40
Total	$49.07

Six scrapers gave the most efficient results and required the constant use of the plow for loosening. The average cost of excavation under these conditions was 25 cts. per cu. yd. The local teamsters were inexperienced in wheel-scraper work, and strongly protested against the use of these machines. The writer believes that the use of the 1-yd. four-wheel scraper would have eliminated this discontent and inefficiency, and reduced the excavation cost to about 15 cts. per cu. yd. This superficial excavation was made with an average cut of 1 ft. at the south and 3 ft. at the north end.

If the hand-excavated material from the footing pits had been shoveled into wagons, hauled away, dumped and then hauled again for the backfill, it became evident from the experience gained in digging the test pits that the cost would have been excessive. Hence, the writer devised the scheme of building wooden bulkheads between the footings. These bulkheads were made about the length of the side of each footing and were placed only on

the transverse spaces between footings. This arrangement left open two continuous aisles or runways longitudinally between the exterior and the interior lines of footing pits (see Fig. 6).

Each bulkhead was composed of two parallel walls of 2 × 12-in. planks, placed with the 12-in. face vertical, and held in position by three bents

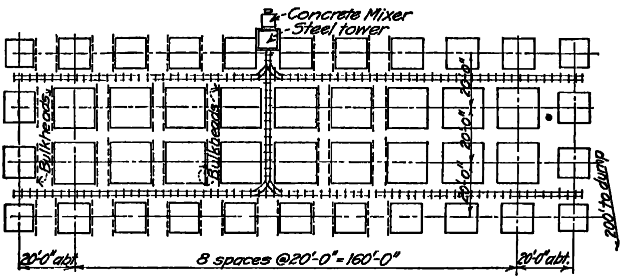


FIG. 6.—Excavation plan of reinforced concrete industrial building.

composed of two 4 × 4-in. posts and cross-bracing at top and bottom. These bents were erected first and the planks added as the excavated material from the pits was piled up. The excavated material was shoveled directly into the bulkheads until they reached a height of about 4 ft. Then it became necessary to shovel the material to the surface at the open sides

FIG. 7.—View of bulkhead method of footing pit excavation.

and reshovel into the bulkheads. The latter were often carried to a height of 8 or 9 ft., and were cross-braced with 4 × 4-in. struts across the pits (see Fig. 7). After nearly a week of continued heavy rains it became necessary to sheath and brace the loaded sides of the pits, as the clay

became saturated with water and began to slide and cave in. This rainy season necessitated a great deal of extra pumping, although the inflow of underground water ordinarily began about 6 ft. below the subgrade elevation. The excavation of the four deep pits at the north end of the site required the removal of a large number of big boulders and necessitated almost constant pumping to remove the inflow of sub-surface water. One gasoline power diaphragm pump and four hand pumps were used.

The excavation for the two end and the following eight interior footing pits was started first and carried down within about 1 ft. of grade during the erection of the pouring tower and mixer plant. As soon as the pouring of the concrete began, the pits were carried down to grade consecutively, so that the concreting closely followed the excavation and both operations became continuous.

The narrow-gage track for the side-dump cradle concrete cars was laid along the two aisles or lanes between the adjacent rows of interior and exterior footing pits. A cross track was also run directly from the tower, connecting the two longitudinal lines of track. The latter was laid on a grade of about 0.5 per cent from the south to the north end of the excavation. As soon as the north footings were completed the surplus material from the southerly pits was loaded into concrete cars, moved down to the completed work and dumped into the pits for backfill.

The excavation of the exterior footing pits, at the north end of the site, was begun as the pouring of the adjacent interior footings neared completion. The excavated material from the upper sections of these side pits was shoveled directly into the center pits of completed footings. All back fill was thoroughly tamped in layers with a heavy iron hand tamper.

The average unit cost of excavation was $0.278 for the wheel scrapers, $0.407 for the drag scrapers, about $0.40 for hand excavation in dry soil and $0.78 for hand excavation in wet soil and rock.

2*d*. Power Shovels.—Power shovels may be classified as to kind of power used, their construction, and method of operation. Recently, the electric motor and the gas engine have been used as the prime mover for the operation of power shovels, but for economic reasons, the steam-operated machines are still generally used.

The customary classification of power shovels is based on their construction and method of operation as follows:

The fixed platform type, where the machinery is mounted on a fixed platform, and the sphere of operation is limited to an arc of about 200 deg. about the head of the machine. This class may be subdivided as follows:

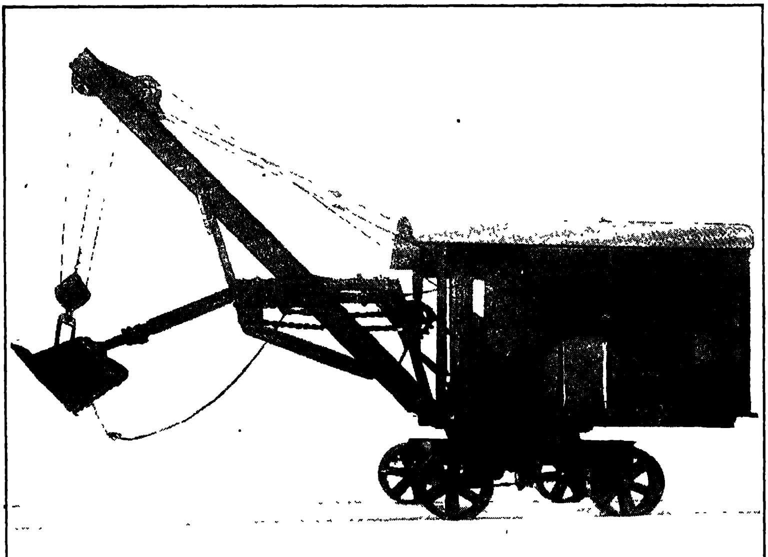
(*a*) Machines mounted on trucks of standard gage.

(*b*) Machines mounted on trucks with wheels other than standard gage.

(*c*) Machines mounted on trucks with small broad-tired wheels.

The revolving class consists of an upper revolving platform on which the machinery is mounted, and the sphere of operation of which is a circle. The upper frame is pivoted on the lower or truck frame, which is mounted on four small diameter, broad-tired wheels for easy mobility over roads.

As the revolving class of power shovel has been designed to meet the present day needs for a power excavator for light work, and is especially adapted for shallow excavation, its construction, operation, and use will be discussed in this article. The fixed platform class is described in Art. **3***a*, p. 50.



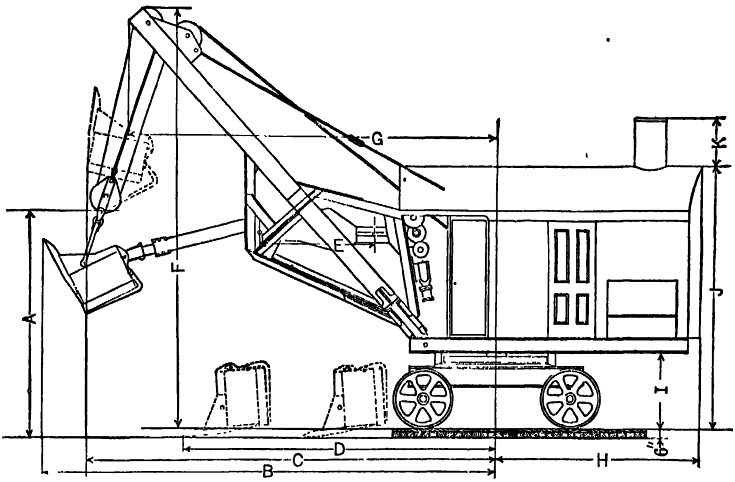
(Courtesy of Thew Shovel Co.)

FIG. 8.—Revolving shovel operated by a gasoline engine.

The revolving shovel may be operated by steam, electricity or internal combustion engine. Steam as the motive power is in general use, but in sections where electric power is cheap or where coal is scarce and high priced, electric power or a gas engine may be economically utilized. Fig. 8 shows the general arrangement of a revolving shovel where a gas engine is used.

The dimensions and working limitations of a well-known make of revolving steam shovel is given in Fig. 9. The Type No., as in column 1 of the table, corresponds to dipper capacities of

⅝, ⅞, 1⅛ or 1⅜, ¾ or 1 (for shale excavation), and 1¾ cu. yd., respectively.



Type No.	A Dumping height	B Max. digging radius	C Dumping radius	D Digging radius at floor	E Length of crowd	F Height of boom	G Swing of boom	H Rear end clearance	I Turntable clearance	J Height of house	K Height of stack	Width of house
A-O	10'0"	21'0"	19'0"	13'6"	5'0"	19'0"	18'6"	9'4"	3'3"	12'3"	2'8"	7'0"
O	10'0"	22'0"	19'9"	15'6"	5'6"	19'0"	19'0"	9'9"	3'6"	12'6"	2'8"	8'0"
O High lift	12'0"	22'9"	20'9"	15'6"	5'0"	22'0"	18'0"	9'9"	3'6"	12'6"	2'8"	8'0"
O Fertilizer	10'0"	18'0"	16'0"	12'0"	6'9"	15'0"	10'2"	5'8"	2'11"	12'0"	.....	7'0"
O Elec. Ry.	10'0"	22'0"	20'0"	15'6"	5'0"	*15'6"	19'0"	7'6"	3'0"	12'0"	.....	7'0"
A-I	10'6"	24'0"	21'6"	18'0"	6'0"	21'0"	19'6"	9'11"	3'10"	13'6"	4'0"	8'10"
A-I High lift	13'6"	27'0"	24'6"	19'6"	6'0"	24'0"	21'6"	10'6"	3'10"	13'6"	4'0"	8'10"
I	10'6"	24'6"	22'0"	18'6"	7'0"	21'0"	21'0"	11'6"	4'2"	14'0"	3'10"	9'2"
I High lift	16'0"	28'0"	24'6"	19'0"	7'0"	26'6"	18'0"	11'6"	4'2"	14'0"	3'10"	9'2"
3	11'0"	28'6"	25'9"	20'6"	8'6"	23'6"	24'0"	12'0"	4'3"	14'6"	3'10"	9'9"

* This dimension 18'6" when boom is extended. *(Courtesy of Thew. Shovel Co.)*

Fig. 9.—Operation limitations of power shovel.

The working capacity of a revolving shovel depends upon the nature of the material to be handled, depth of cut, efficiency of hauling equipment, efficiency of operator, size, capacity and

efficiency of shovel, etc. In ordinary clay and loam or sandy clay, under average working conditions, with a cut of from 5 to 10 ft., the output for a 10-hr. day should average from about 500 cu. yd. for a ⅝-yd. machine to 1000 cu. yd. for a 1¾-yd. machine.

Field of Use.—The power shovel has become one of the most efficient and universally used of excavators where the magnitude of the work and soil conditions warrant its use. It can excavate all classes of soil from loam to loose rock, and also solid rock after it is broken up by blasting.

The revolving shovel can be used to advantage on the excavation of cellars or basements for buildings, on excavation for the

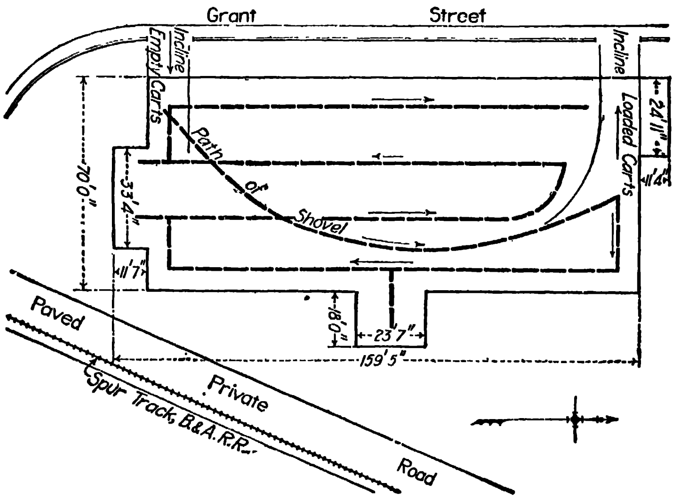


FIG. 10.—Diagram of steam shovel operations in cellar excavation.

foundations of walls and dams, and also for excavation for other structures where the amount of material to be handled justifies the transportation and operation costs. Under such conditions, the use of a power shovel will often affect a saving of about 50 per cent over hand shoveling.

A steam shovel was used, during the summer of 1914, in excavating the basement for a reinforced concrete factory building for the Dennison Manufacturing Company, at South Framingham, Mass. The building is rectangular in form, 70 ft. × 159 ft. 5 in., with two projecting stair towers and a toilet tower (see Fig. 10).

The soil excavated was a fine, clean, siliceous sand, in beds 3 to 7 ft. in depth, and separated by strata of yellow clay of a depth of 1 to 2 ft. The

excavation was carried down to a gravel subsoil, upon which the footings were placed. The depth of excavation varied from 8.2 to 10.5 ft. The material was shoveled into rear end dump carts and hauled to two low, swampy tracts of land located about ⅓ mile from the building site.

The bulk of the excavation was made with a Thew automatic revolving steam shovel, Type O, equipped with a ⅝-yd. dipper. The shovel began operations near the southwest corner of the building plot, and excavated a cut about 15 ft. wide on a descending grade of about 10 per cent. As the shovel approached the northeast corner of the plot, it reached the finished grade, about 10.5 ft. below the original ground surface.

On account of the loose character of the soil and the inflow of water where the excavation reached grade, it was necessary to support the shovel on planking. A sectional, movable platform was built of 4 × 8-in. timbers, bolted together to form sections 3 ft. wide and 12 ft. long. Four of these sections were used on straight stretches, and two triangular-shaped sections, half the size of the rectangular sections, were used on the turns.

Neglecting time lost through breaks in machinery, inclement weather, etc., the shovel was excavating about 60 per cent of the working time. Special effort was made to keep the shovel always supplied with wagons, and very little time was lost in waiting for wagons. From two to three shovelfuls were required to load each wagon to an average capacity of 1⅓ cu. yd. (loose measurement). The average time to load a wagon, with three swings, was 1 min. 46 sec. and the minimum time was 1 min. 21 sec.

Eight to fourteen teams were used for the transportation of the excavated material over an average length of haul of 1800 ft. The teams were run over a continuous circuit and the loading, hauling, and dumping were supervised to eliminate delays and "bunching." The use of a bonus, on a sliding scale, increased the hauling efficiency about 30 per cent.

The average daily yardage handled over a working period of fifteen 9-hr. days was 359 cu. yd., the minimum being 213 cu. yd. and the maximum 461 cu. yd.

During 1912, an electrically operated Type No. 0 Thew shovel was used for the basement and foundation excavation for the new City Hall in Boston, Mass. The project required the excavation of an area of 225 × 50 ft. to a depth of 20 ft. Loose clay was found near the surface, but rocks were encountered at a slight depth. The excavated material was hauled away by thirty 1½-yd. rear dump wagons, which had access and egress to the excavation over a steep incline. Each wagon made an average of 11 to 12 trips per 9-hr. working day. The average loading time per wagon was about 2 min., while it required 15 min. for three men to load a wagon by hand shoveling.

The electric shovel operated in Boston clearly demonstrated the advantage of the use of electric power over steam power in the elimination of smoke, water line, hauling and handling of fuel, a fireman, a watchman, the danger from freezing, and the discomfort of boilers in hot weather.

2e. Scraper Excavators.—Scraper excavators may be classified as to their method of operation—namely, stationary machines with a pivoted boom, and revolving machines.

The form of stationary machine with a pivoted boom which can be utilized on general construction work is very similar to the small power shovel. This machine has a fixed platform supported on small diameter, broad-tired wheels for easy portability, and carrying the power and excavating equipment, consisting of boiler, hoisting, and swinging engine, A-frame, boom, dipper-handle, and dipper or bucket. As in the fixed

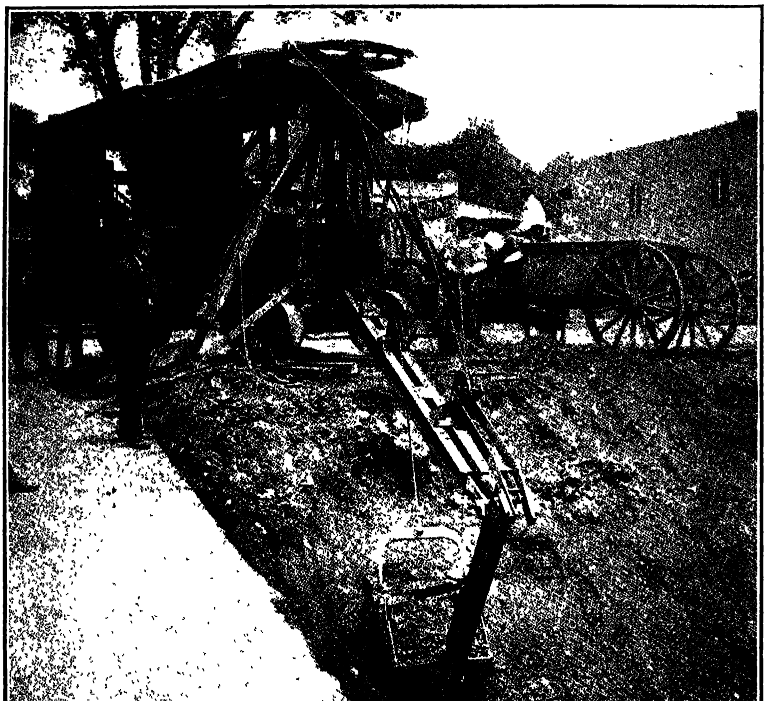


Fig. 11.—Scraper excavator on cellar excavation.

platform type of power shovel, jacks or spuds are located on either side of the platform near its front end to prevent tipping during the side swinging of the boom. The bucket used is shaped like a scoop and is drawn towards or shoved away from the machine. A horizontal steel-framed arm serves as a guide for the scoop or bucket. Fig. 11 illustrates this form of excavator.

The revolving type of scraper-bucket excavator is becoming a universally useful machine for excavation under conditions unsuited for power shovel operation. This machine may be

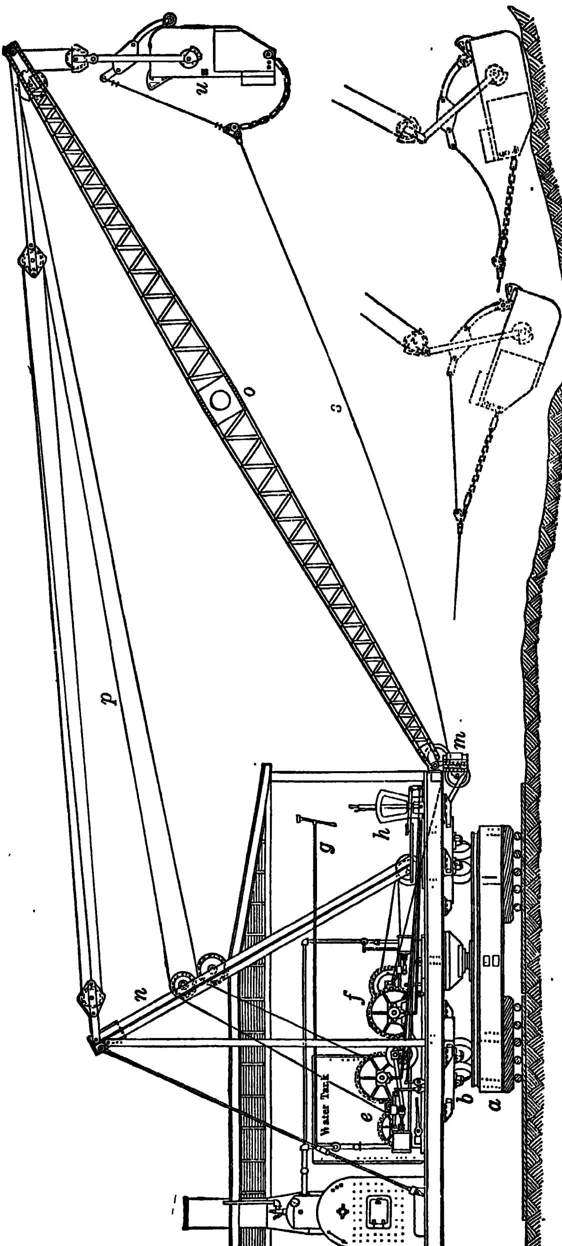
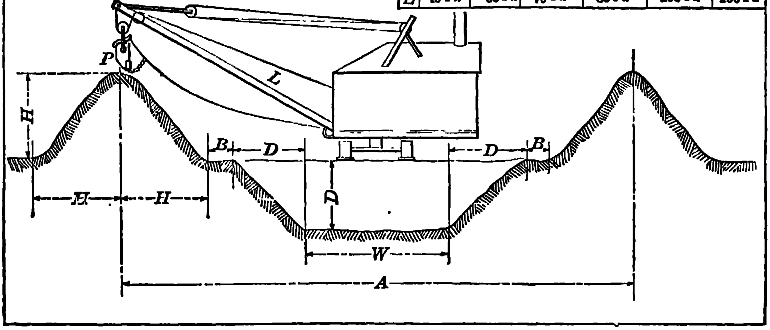


FIG. 12.—Detail view of drag-line excavator. *a*, Lower frame; *b*, turntable; *c*, upper frame; *d*, boiler; *e*, swinging engine; *f*, hoisting for main engine; *g*, throttle lever; *h*, engine levers; *m*, fair lead pulleys for hauling cable; *n*, A-frame; *o*, boom; *s*, hauling line; *u*, bucket.

mounted in one of three different ways: (1) On skids and rollers, (2) on trucks, and (3) on caterpillar tractors.

The essential parts of a scraper-bucket or drag-line excavator are the substructure (which consists of the upper and lower platforms and turntable), the power equipment, the hoisting engines, the swinging engines, A-frame, boom, and bucket. These essential parts, their system of co-ordination, and their method of operation are practically the same in all types and makes of this class of excavator and differ only in details of construction. The principal parts of a scraper-bucket excavator are shown in Fig. 12.



	Type No.1	Type No.2	Type No.3	Type No.4	Type No.5	Type No.6
A	76′0″	128′0″	142′0″	168′0″	184′0″	198′0″
B	5′0″	8′0″	8′0″	10′0″	10′0″	10′0″
D	10′0″	12′0″	12′0″	14′0″	15′0″	15′0″
H	13′0″	20′0″	21′0″	25′0″	27′0″	29′0″
W	20′0″	50′0″	60′0″	70′0″	80′0″	90′0″
P	1 Cu. Yd.	2 Cu. Yd.	2¼ Cu. Yd.	2½ Cu. Yd.	2½ Cu. Yd.	3 Cu. Yd.
L	40 Ft.	60 Ft.	70 Ft.	80 Ft.	100 Ft.	100 Ft.

Fig. 13.—Operation limitations of drag-line excavator.

Field of Use.—The application of the drag-line principle permits of the excavation of material at a considerable depth below the surface and its elevation to a corresponding height above the surface. Hence, the scraper-bucket machine can operate to advantage from the surface in the removal of soft, wet, soil in a basement or pit. Fig. 13 shows the limitations of operation for various sizes of a well-known make of drag-line excavator.

The use of the stationary form of scraper excavator with pivoted boom in basement excavation was demonstrated on a job at Kankakee, Ill. in 1916.[1] The work comprised the excavation of an area 125 ft. long by 95 ft. wide,

[1] From *The Contractor*, April 1, 1916.

with an average depth of 10 ft. and a maximum depth of 12 ft. The material was a yellow clay with some broken rock, and the total yardage was about 4200 cu. yd. The machine used was a Keystone 10-ton traction shovel, which was especially equipped with a scoop-arm pivoted to the end of the horizontal guides. The scoop was hinged to the lower end of the handle and filled by being drawn towards the machine by a cable on the hauling drum. The machine operated entirely from the surface and loaded wagons drawn up alongside, thus doing away with the necessity of operating the machine in the excavation and hauling the loaded wagons out with extra power. The excavator was served with 13 wagons, the average loading time of which was 2 min. 19 sec. Observation showed that about 5 additional wagons could have been efficiently used.

The unique use of a special form of scraper-bucket excavator is illustrated in the excavation of the basement of the Wisconsin Food Products Company of Milwaukee, Wis.[1] The building was started in November, 1919, and

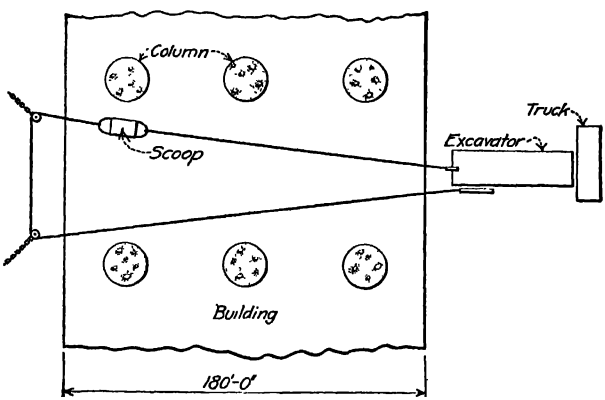


FIG. 14.—Diagram of operation of scraper excavator in cellar excavation.

only sufficient excavation made in the frozen soil to install the footings for the reinforced concrete columns. The following Spring a Smith excavator and loader was used to remove the soil between the columns. Fig. 14 shows the arrangement of machinery and method of operation. As shown in the figure, a wire cable passed from the hoisting drum on the excavator to the scoop, and then through the sheaves to the back-haul drum. With an insufficient number of trucks for the removal of the excavated material, the output averaged 300 cu. yd. per 10-hr. working day. On this job, the space between the ground surface and the bottom of the first floor was less than 5 ft. and prohibited the use of a steam shovel.

2f. Cableway.—The drag-line cableway, as developed and used on the Chicago Drainage and New York State Barge Canals, consisted essentially of a tower or mast which was either fixed or movable, the operating equipment and excavating

[1] From *Engineering and Contracting*, June 16, 1920.

equipment. Fig. 15 shows the general features of a drag-line cableway, with a tower movable on a portable track, the operating equipment consisting of a vertical boiler and double-drum

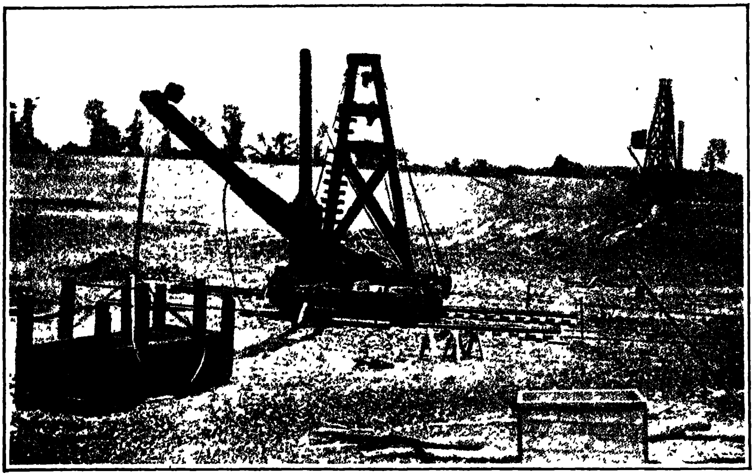


Fig. 15.—Drag-line cableway excavator.

hoisting engine and the simple, two-line scraper bucket. More complex forms of automatic and self dumping buckets have come into use in recent years, as is shown in Fig. 16, which

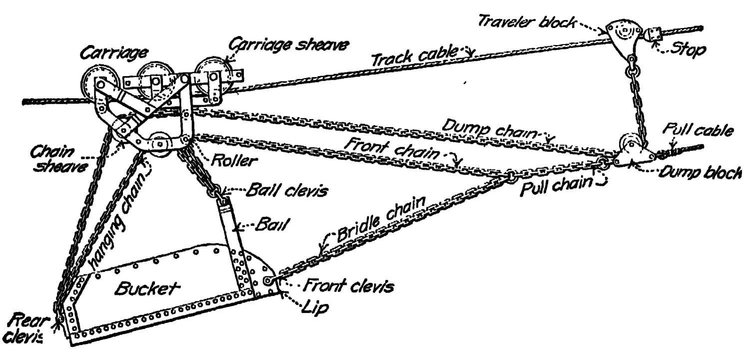


Fig. 16.—Drag-line cableway bucket.

automatically dump by the pull on the dump chain, when the carriage is stopped by the stop set on the track cable.

A double tower, one set on each side of the excavation with a bucket moving in two opposite directions has been used to advantage in wide, shallow excavations.

Field of Use.—A single tower excavator equipped with a 75-ft. tower, a 2-yd. scraper-bucket and a 10 × 12-in. double drum, vertical hoisting engine, and operating over a width of 250 ft., should be able to average an output of 600 cu. yd. in a 10-hr. day.

Cableways with towers moving along curved tracks were used in 1914 for the excavation and embankment construction for the Yale "Bowl" at New Haven, Conn. The excavation comprised the removal of about 175,000 cu. yd. for the bottom of the bowl and the lower 29 rows of seats. The material was black and yellow loam to a depth of 22 in. and underlaid with sand and gravel. Two 5/8-yd. steam shovels and two drag-line excavators were used for the work. The drag-line machines consisted each of a wooden tower 85 ft. high and 50 ft. square at the base, which moved along a double elliptical track outside of the bowl; operating machinery consisting of a two 12 × 16-in. cylinder, Lambert cableway engine and a 125-h.p. locomotive boiler; and excavating equipment of a 2-yd. scraper-bucket propelled by a 7/8-in. plow steel cable. The average daily excavation was 400 cu. yd. and the maximum excavation was 3000 cu. yd. during a 24-hr. period. The maximum distance from tower to deadman was 750 ft. and the corresponding greatest excavation was 1500 cu. yd. for a 24-hr. day. The maximum monthly excavation was 44,400 cu. yd.

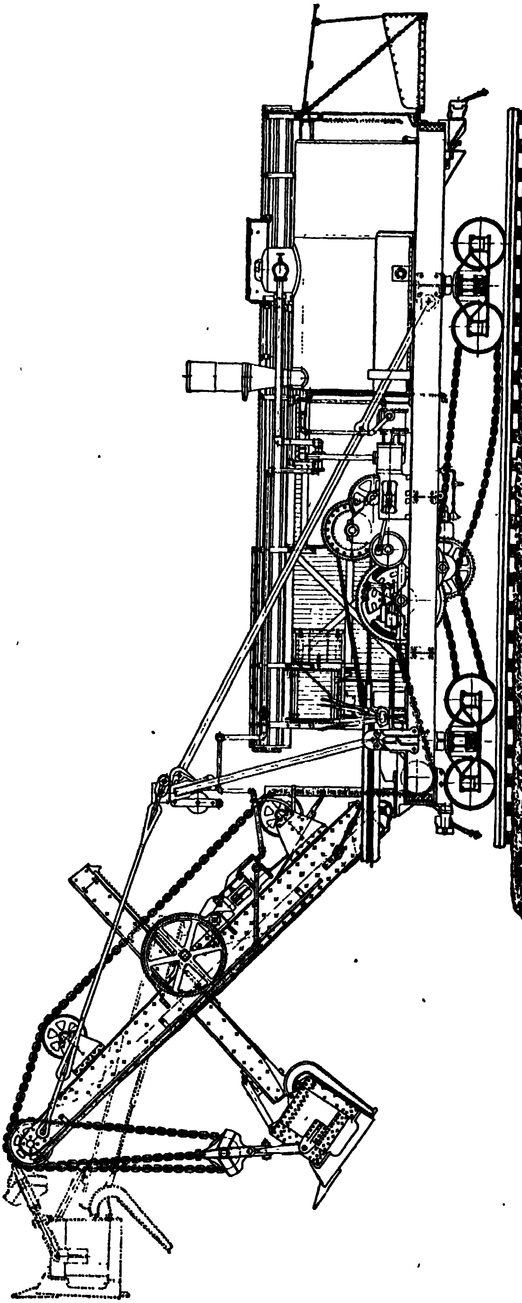
3. Deep Excavation.—This class of excavation includes the basement excavation for tall buildings, deep foundations for bridge abutments, walls and dams, deep pits, etc. Generally, hand tools can only be economically used in this class of work if the area is too restricted for power machinery to work. Whenever space permits, some type of power excavator should be used in order to secure maximum output and greatest economy, especially as to labor cost. Hand excavation is ordinarily about twice as expensive as power shovel work.

Deep excavation involves the use of machinery with a large range of vertical action in the handling of the material, or auxiliary hoisting and transporting machinery. For example, in the excavation of a large dam foundation, scraper-bucket excavators working from the surface might be able to handle all the material to a depth of 25 or 30 ft. For the greater depths steam shovels operating in the excavation would load skips or buckets, which would be lifted to the surface by a derrick or cableway.

Excavation to a considerable depth generally involves sheeting, bracing, and shoring in earth and loose rock, and blasting in solid rock. Methods of sheeting and cofferdam construction is discussed in the chapter entitled "Cofferdams," and blasting is described in Art. **4.** This article will take up only the use of various types of power excavators.

TABLE 3.—SIZE OF A STANDARD STEAM SHOVEL

Type		Chain						Wire rope	
Class		110C	100C	85C	70C	60C	40R or C	80	45
Effective pull on dipper (lb.)		98,000	91,000	70,000	64,000	56,000	33,000	80,000	45,000
Capacity of dipper (cu. yd.)		3½ to 6	3½ to 5	3 to 4	2½ to 3	2½	1½	3 to 5	2½
Size of engines (double cylinder)	main	13″×16″	12½″×16″	12″×15″	10″ ×14″	10″ ×12″	8″ ×8″	12″×12″	10″×10″
	swing	9″× 9″	8″ × 8″	8″× 8″	7½″× 7″	7½″× 7″	5½″×6″	9″× 9″	7″× 8″
	thrust	9″× 9″	8″ × 8″	8″× 8″	7½″× 7″	7½″× 7″	5½″×6″	9″× 9″	7″× 8″
Car body	length	44′ 9¾″	44′ 2″	41′ 4″	36′ 4½″	35′	26′ 5⅜″	42′	36′
	width	10′	10′	10′	10′	9′ 3″	7′	10′	10′
Wheel base	traction	....	....	....	....	....	16′ 2″		
	truck	35′ 6″	35′ 10½″	33′ 5″	30′ 3½″	28′ 9″	21′ 1½″	36′	31′
Width over traction wheels		....	....	....	....	....	15′		
Height of A-frame	extreme	20′ 7¾″	19′ 3″	19′	19′	18′ 10″	14′ 1½″	21′ 8″	19′ 6″
	lowered	14′ 6″	14′ 6½″	14′ 6″	14′ 6″	14′ 6″	....	15′	15′
Boiler	type	Loco.	Loco.	Loco.	Loco.	Loco.	Loco.	Loco.	Loco.
	dimensions	58″×18′ 3″	54″×18′	50″×18′	44″×18′	44″×17′	42″×13′ 6″	52″×21′ 4″	46″×20′
Water tank capacity (gallons)		1,800	1,600	1,600	1,500	1,500	700	2,000	1,950
Weight in working order (tons)		130	113	101	87	77	48	101	73
Shipping weight	domestic (tons)	116	101	89	75	65	42	88¾	64½
	export, boxed, approximate (gross tons)	121	104	91¾	77¾	67	96	96	71½



(Courtesy of the Marion Steam Shovel Co.)

FIG. 17.—Fixed platform type of power shovel.

3a. Power Shovel.—Deep excavation, often requiring the handling of the denser and harder soils, such as rock, generally requires the fixed platform type of power shovel. Hence, this article will supplement Art. **2d,** and describe this class of excavator. On work of small magnitude and in restricted working space, the revolving shovel may be the best adapted to the job and should be used.

All three types of the fixed platform shovel differ largely in their method of support, but otherwise are similar in their

	Class 80-18-4½	Class 60-17-3½	Class 45-16-2½	Class 25-11-1¼
A	33′0″	30′0″	27′0″	20′0″
B	18′6″	17′6″	16′6″	11′6″
C	11′6″	10′6″	9′6″	4′6″
D	16′6″	15′6″	13′6″	7′0″
E	32′0″	30′0″	26′6″	21′0″
F	19′0″	18′0″	18′0″	13′0″
G	3 to 5 cu. yds.	3½ cu. yds.	2½ cu. yds.	1¼ cu. yds.
H	5′0	5′0″	4′0″	2′6″

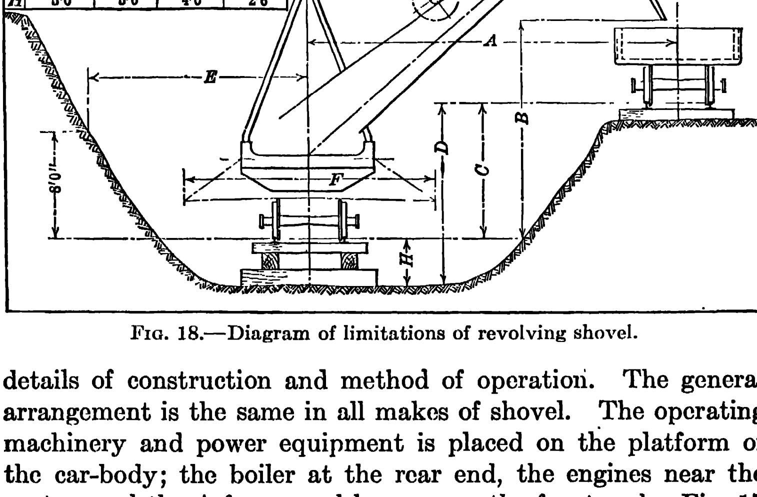


Fig. 18.—Diagram of limitations of revolving shovel.

details of construction and method of operation. The general arrangement is the same in all makes of shovel. The operating machinery and power equipment is placed on the platform of the car-body; the boiler at the rear end, the engines near the center, and the A-frame and boom near the front end. Fig. 17 shows the general construction of a steam shovel of the fixed platform class.

The sizes of a well known make of steam shovel are given in Table 3.

The dimensions and working limitations are shown in Fig. 18 and Table 4.

The steam shovel of the fixed platform class can excavate any material except solid rock, which must first be blasted and broken up into pieces small enough for the dipper to handle. The excavated material may be dumped into and carried away

TABLE 4.—WORKING LIMITS OF A FIXED-PLATFORM SHOVEL

Type	Chain						Wire rope	
Class	110C	100C	85C	70C	60C	40C or R	80	45
Dumping radius, *A*	32′	29′	29′	27′	25′	21′ 6″	33′	27′
Height of dump, *B*	17′	17′	16′ 6″	16′ 6″	16′	12′	18′ 6″	16′ 6″
Depth of cut, shovel track to loading track *C*	10′	10′	9′ 6″	9′ 6″	9′	5′	11′ 6″	9′ 6″
Maximum depth of through cut, *D*	16′	15 6″	15′	14′	13′	7′ 9″	16′ 6′	13′ 6′
Digging radius 8 ft. Elevation *E*	33′	33′	33′	30′	27′	23′	32′	26′
Spread of jack screws, *F*	..	22′	20′	18′ 4″	18′	15′	19′	18′
Height of boom, *G*	..	33′	28′ 9″	29′ 1″	27′ ½″	26′ 9″	33′	27′ 7″
Depth of cut below rail, *H*	..	6′	5′ 6″	4′ 6″	4′	2′ 9″	5′	4′

by: (1) dump wagons, hauled by teams or by traction engines; (2) self-propelled motor trucks; (3) dump cars holding from 1½ to 6 cu. yd. drawn by horses or dinkey locomotives over narrow gage track; and (4) dump cars of large size, from 4 to 12 cu. yd., gondola or flat cars hauled by large size locomotives over standard-gage track.

The output of a power shovel depends on its size, the character fo the material to be excavated, the efficiency of the crew, climatic conditions, location of material with relation to shovel, relation of shovel to point of dumping, efficiency of wagon or car service, etc. The maximum working capacity of a shovel, under favorable conditions, will be about one-half of its theoretical rated capacity. A shovel is ordinarily in actual operation about 40 per cent of the working time. The balance of the working time being taken up by delays for repairs, coaling, oiling, watering, cars, etc. The log of efficient shovel operation under favorable working conditions would be about as follows:

OPERATION	TIME PER CENT
Moving shovel	10
Breaking up rock, mucking, etc	10
Waiting for cars or wagons	15
Repairs	60
Actual loading	5
Total	100

Table 5 gives an idea of power shovel output, and is based on records of the operation of about 50 shovels during a period of several weeks.

TABLE 5.—STEAM SHOVEL OUTPUT[1]

Division	Shovel size (tons)	45	55	65	70	75	90	95	Summary
Iron ore	Observations	...	...	....	7	...	1	1	9
	Maximum	...	...	....	1,512	...	2,728	1,350	2,728
	Minimum	...	...	....	892	...	2,728	1,350	2,728
	Average	...	...	....	1,095	...	2,728	1,350	892
Sand and gravel	Observations	2	...	....	3	...	....	....	5
	Maximum	373	...	....	3,300				
	Minimum	360	...	....	1,602	...	....	....	360
	Average	366	...	....	2,365	...	....	....	1,506
Earth and glacial drift	Observations	...	...	1	3	...	....	1	5
		...	...	1,065	1,426	...	....	1,073	1,426
		...	...	1,065	569	...	....	1,073	569
		...	...	1,065	893	...	....	1,073	963
Rock	Observations	...	...	5	16	...	....	5	26
	Maximum	...	...	896	1,542	...	....	1,200	1,542
	Minimum	...	...	264	168	...	....	154	154
	Average	...	...	601	682	...	....	873	704
Clay	Observations	...	1	2	5	1	....	1	10
	Maximum	...	320	780	1,415	820	....	990	1,450
	Minimum	...	320	474	498	820	....	990	320
	Average	...	320	627	1,064	820	....	990	870
General summary	Observations	2	1	8	31	1	1	8	55
	Maximum	373	320	1,065	3,300	820	2,728	1,350	3,300
	Minimum	360	320	264	168	820	2,728	154	168
	Average	366	320	665	991	820	2,728	972	934

Field of Use.—The power shovel can be universally used where the soil is firm enough to support it, the working area is sufficient and the size of the job warrants its use. In excavation of deep foundations for large dams, walls and reservoirs, the power shovel can often be efficiently employed in conjunction with hoisting machinery such as derricks and cableways. In the construction of embankments or dams for reservoirs, the material may often be removed from pits near the ends of the structure and transported in trains of dump cars to the site. For small work (where the amount of material handled may be less than 100,000 cu. yd.), the revolving shovel and dump wagons or motor trucks may be economically used.

[1] **Prepared by R. T. Dana, Construction Service Co., New York.**

In 1910–12, a fixed platform shovel was used in the excavation of material for the embankments forming the northerly and southerly ends of the Aziscohos Dam near Colebrook, N. H. The shovel worked in a borrow pit on a hillside about 500 ft. from the embankment, and the slope to the latter permitted gravity transportation, by carts, of the excavated material. The latter consisted of a hard, compact glacial clay, locally called rock flour, with about 6 per cent of large boulders and a small percentage of small stone. The total amount of material excavated by the shovel was 23,614 cu. yd. The maximum daily output was 408 cu. yd. in 11 hr. About 5000 cu. yd. of material was placed by hand derricks and skips, the double shoveling costing $1.15 per cu. yd.

During part of 1912, an Atlantic steam shovel equipped with a 2½-yd. dipper, excavated about 57,700 cu. yd. of loose boulders, earth and rock, in the preparation of the foundation of the Arrowrock Dam near Boise. Idaho. The transportation equipment consisted of two Vulcan 16-ton and one American 16-ton locomotives and twenty-five 4-yd. Western dump cars. An interesting feature of the work was the handling of occasional pockets or nests of boulders ranging up to 150 cu. yd. in size. Considerable blasting was required and the shovel often put to hard usage. From Feb. 27, 1912 to Oct. 10, 1912, the shovel removed 27,300 cu. yd. of solid rock and 30,400 cu. yd. of other material at an operating cost of 25 cts. per cu. yd. The transportation cost was 24 cts. per cu. yd.

3b. Scraper-bucket Excavator.—The general features of construction and operation of the drag-line or scraper-bucket excavator are described in Art. 2e. This form of excavator, equipped with steam, gasoline, or electric power and supported on rollers, wheels, or caterpillar tractors is very adaptable as regards local conditions of fuel supply and soil conditions. Its wide flexibility of operation make this machine especially efficient in deep pit and foundation work.

An efficient use of the drag-line excavators was shown (1921) in the excavation of the borrow pit area for the construction of the Englewood Dam of the Miami valley flood protection works.[1] Two 115-ton drag-lines, one steam and one electrically operated, equipped with 85-ft. booms and 4½-yd. buckets, operated the coarse glacial gravel and loaded it into five-car trains of 12-yd. cars. The machines moved along loading tracks 150 ft. apart running north and south across the area, and connected with cross-tracks and spurs to the hog-box sumps, where the material was dumped and then pumped into the dam embankment. The two machines were spaced about 1500 ft. apart and moved simultaneously along each cut, so that one machine reached the middle of the cut as the other reached the end. The average monthly output for each machine was 75,000 cu. yd., and the maximum daily output, based on two 10-hr. shifts, was 2740 cu. yd. It is interesting to note that experience with two million cu. yd. of excavation shows a shrinkage of one-third where placed in embankment.

[1] From *Eng. News-Record*, April 7, 1921.

3c. Locomotive Crane.—The locomotive crane or traveling derrick is a very serviceable type of excavating, hoisting, and conveying machine. The essential parts of a crane are the car, the hoisting engine, and the derrick. The machines are made in capacities of from 3 to 20 tons. The construction of a typical machine is shown in Fig. 19. The power for cranes may be steam, electric, or that furnished by an internal combustion engine. Ordinarily, steam power is used, but the other

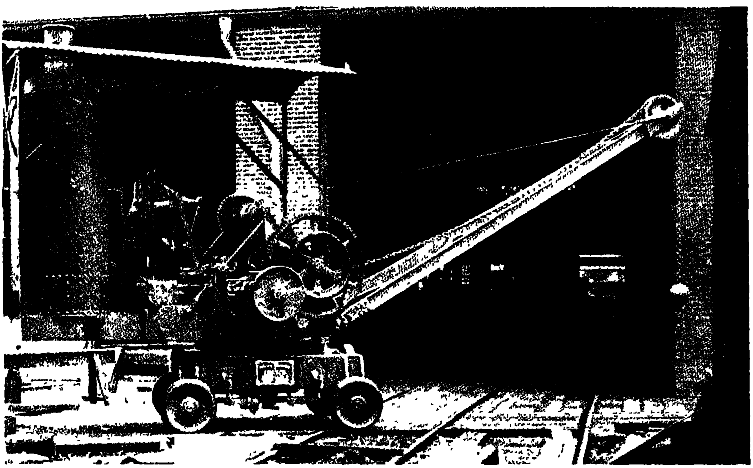


Fig. 19.—Locomotive crane.

kinds would be more economical where electric power is cheap, or coal and wood is expensive as compared with electricity and gasoline.

The excavating element, consisting of the bucket or dipper, may be a grab bucket of the orange-peel or clam-shell type, or a drag-line dipper. The former is used for excavation of the looser and softer soils—such as loam, sand, and sandy-clay—while the latter is necessary for the removal of the harder and denser soils as gravel, dense clay, and loose rock. The capacities of the orange-peel bucket (see Fig. 20) varies from ½ to 2 yd., while the clam-shell bucket is made in capacities of from ½ to 3 yd. (see Fig. 21).

Field of Use.—The locomotive crane is especially adapted to the excavation of foundation trenches where the soil conditions are favorable for the use of a grab or scoop bucket. The machine

is also useful in backfilling behind a wall and for the removal of sheeting and bracing. A 10-ton machine equipped with a 1-yd. clam-shell bucket and moving on a track alongside a foundation trench, will excavate loam and glacial clay to a depth of about 8 to 10 ft. at the rate of about 400 cu. yd. per 10-hr. day.

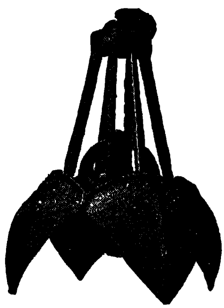


Fig. 20.—Orange-peel bucket.

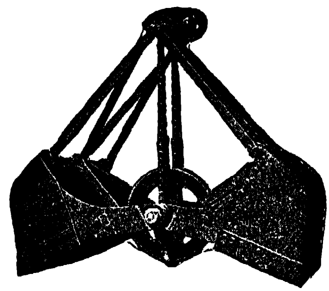


Fig. 21.—Clam-shell bucket.

During the latter part of 1916 and the early part of 1917, a locomotive crane with a 1-yd. clam-shell bucket and a Lidgerwood hoist operating a derrick with a skip from a 35-ft. boom, were used in the excavation of foundation trenches for about 1950 ft. of 4-ft. concrete retaining walls about the site of the Bevo plant of the Anheuser-Busch Brewing Co. at St. Louis, Mo.

3*d*. **Fall-line Cableway.**—The fall-line cableway is especially adapted for the transportation of material across a great open space, such as a valley or stream, the hoisting and removal of excavated material, and the excavation of pits, trenches, and foundations.

The principal parts of a cableway are the towers, the power equipment, and the operating equipment.

The terminals of a cableway are generally wooden-framed towers which may be arranged in one of three ways: (1) Two fixed towers, (2) one tower fixed and the other mounted on a barge in water or traveling on a circular track, and (3) two movable towers traveling on parallel tracks.

The power equipment generally consists of a steam boiler of the vertical tubular type, and a two-drum, double-cylinder, reversible link-motion engine. Electricity and compressed air have been used with success in the operation of cableways, especially where electric power was available and cheap in cost.

The operating equipment comprises a traveler, tubs, buckets or skips, and the cables. The main cable serves as a track over

which the traveler moves and is controlled by hoisting and traversing cables operated from the engine. Figure 22 gives a general view of a fall-line cableway.

Field of Use.—The cableway has an important field of usefulness in the excavation of deep foundations for dams, piers, and walls. The material may be blasted or hand excavated and

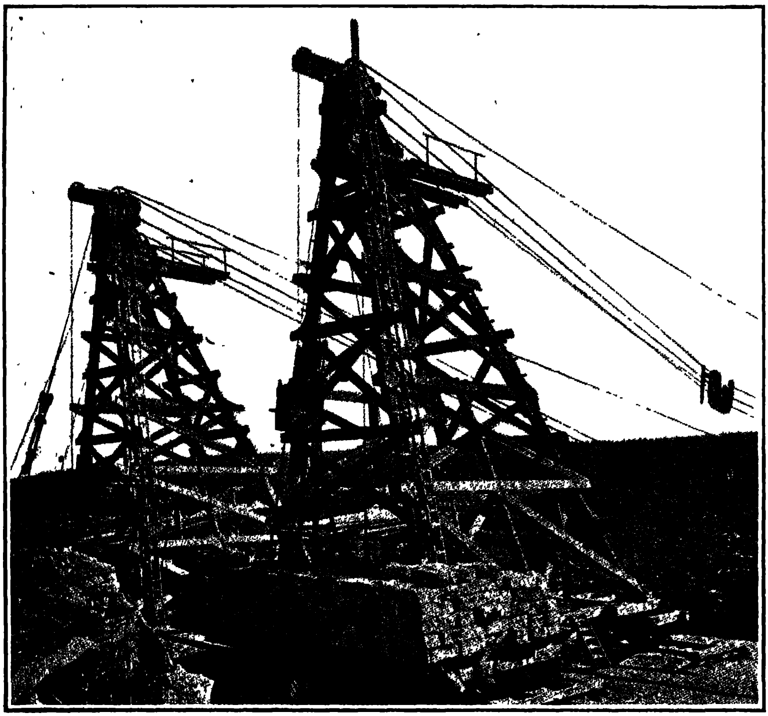


Fig. 22.—Fall-line cableway on dam construction.

loaded into skips by hand or power shoveling. The loaded skips are raised and transported to the dump or to cars, trucks, or wagons for further removal. On large trench excavation, where the trench is over 6 ft. wide and 10 ft. deep, this machine can operate efficiently. Such an excavator with two 30-ft. towers, operating 1-yd. buckets, should handle 300 cu. yd. of clay and gravel per 10-hr. day. A crew of 30 men would be required to excavate the material and load the buckets.

3e. Derrick and Hoist Buckets.—Derricks may be of various types and sizes, from the small, light weight, hand-operated machine to the large sized stationary or movable type. The former are often useful in the excavation of narrow, restricted but deep excavations, such as pits and trenches in the city, while the latter are best utilized in deep trench work in the open country.

The derrick consists of a platform upon the front end of which is mounted the leg or boom, from the outer end of which is suspended the skip or bucket. Upon the rear of the platform is placed the operating equipment, generally consisting of a steam boiler and vertical hoisting engine. In localities where electricity is cheap, a motor operated hoist can be used economically, thus doing away with the labor and bother of handling coal, ashes and water.

Two classes of buckets may be used with a derrick—the non-digging dump bucket, and the digging or grab bucket. The first class include skips, trunnion buckets, and bottom-dump buckets. The second class comprises the orange-peel and the clam-shell buckets. The skip is a tray-shaped box with one side open. It may be made of wood or steel and is suspended from three points by chains leading to a ring which engages the hook on the end of the derrick cable hoist line. The trunnion bucket consists of a steel tray, the front side of which slopes sharply toward the outer or upper edge. The bucket is supported from the hoist line by a bail which is attached to the sides by trunnions so placed as to keep the bucket upright when loaded. The bucket is easily dumped by tilting. Bottom dump buckets are steel boxes with the bottoms hinged so that their contents may be automatically dumped. This type of bucket is better suited for handling concrete than earth and possesses few advantages over the skip for the removal of hand excavation.

The movable or traveling derrick consists of a derrick, the platform of which rests upon rollers or wheels. This type of excavator has been described in Art. **3c**.

Field of Use.—The fixed, stiff-leg derrick is especially adapted for the hoisting of material from pits and deep foundation work. Skips or buckets loaded by a power excavator or by hand shoveling, may be hoisted to the surface and dumped into cars, wagons, or trucks. This method of elevating excavated earth

and rock is often used in deep foundations for dams, walls, piers, and buildings.

The movable derrick is utilized to good advantage in trench excavation, where the platform of the machine moves along the side of the trench, or in the case of narrow trenches, straddles the trench.

An example of foundation excavation with a stiff-leg derrick occurred in the excavation of the foundation of the plant of the French Bros.—Bauer Co. of Cincinnati, O. in 1917.[1] The derrick was located near the center of the site and consisted of a 60-ft. boom, a 40-ft. mast, and a ¾-yd. Owens bucket operated by a Drake swinging engine. The bucket discharged into wagons on an elevated bin to secure continuous operation. The output averaged 28 yd. per hr. at a labor cost of 48 cts. per cu. yd.

The east end of the site required excavation to a depth of 30 ft. below the street grade. For this work, the contractors rigged up a drag-line skip which was hauled up an inclined runway composed of a timber frame supported against an old building wall. The skip had a ½-yd. capacity and at the top of the runway, dumped into wagons on the street underneath. The material was loam and sand and was removed at a cost of 32 cts. per cu. yd.

4. Rock Excavation.—The excavation of deep foundations for buildings, walls, dams, piers, and other similar structures often involves the removal of solid rock. The rock must be broken up into fragments of sizes that can be handled and removed, either by hand or power excavators. Blasting is universally used as the method of breaking up rock, and consists in the drilling of holes into the rocks, the charging of the holes with a suitable explosive, and the firing or explosion of the charge. As the breaking-up of rock in the foundation of many structures such as buildings, piers, and walls must often be done in restricted areas, and adjacent to existing structures, the work must be executed with great care to secure the required results without injury to life and property.

Rock drilling may be done by hand or by machinery. Hand drilling is not economical unless the amount of work is very limited (restricted area and short depth of holes) or power (steam, compressed air, and electric) is not available. Ordinarily some form of power drill should be used. In limited areas and for shallow work, the small portable, light types of drills are preferable, while on large jobs with deep holes, the larger machines of the cable type are better adapted.

[1] From *Engineering and Contracting*, July 18, 1917.

4a. Hand Drilling.—The drilling of holes in rock by hand may be done by a rotary drill or auger, a hammer drill, or a churn drill. The auger is used only in soft rock such as shale, coal, or peat.

The hammer drill consists of a steel rod of from ¾ to 1½ in. in diameter, provided with a specially formed and hardened cutting edge of from ⅜ to ½ in. larger than the rod. The shape of the cutting edge depends on the use to which it is to be put; a chisel bit being ordinarily employed, while rose and star bits are especially adapted for drilling holes in brick and concrete masonry. The cutting edge is rounded on the bottom to ensure effectiveness in cutting and to allow for the tilting of the drill.

Hand drilling may be done by one or more men, depending on the depth of hole. For holes up to 3 ft. in depth, one man with a 4½-lb. hammer can work economically. For greater depths of hole, a heavier hammer is necessary. One man should operate the drill and a second man strike with a 10-lb. hammer. In hard rock and for deep holes, two strikers can be used to advantage.

The hole should be started on a solid and leveled surface with a short drill. Light blows should be struck at first, and the drill turned one-eighth of a revolution after each blow. Water is poured into the hole periodically to hold the powdered rock in suspension. A "spoon," a ¼- to ½-in. rod with a disc on one end, is used to remove the paste from the hole. The latter may also be cleaned by waste fastened to the end of a wooden bar or stick.

Churn drilling consists in operating a drill rod with a special bit (generally a double bit) in such a manner as to utilize the falling and twisting of the drill in fracturing and pulverizing the rock. The churn drill often comprises a pipe or rod, on one or both ends of which the bit section, having a length of from 12 to 18 in., is welded. For short drills of small diameter, the weight is increased by welding a ball of iron to the center of the drill shank, making a ball drill. The ball drill may be operated by one man, while the larger drills for the drilling of deeper holes are generally operated by from two to six men. Wooden bars are clamped or bolted to the drill to serve as handles for its lifting and operation.

Scope of Work.—The number of feet of hole that can be drilled varies with the diameter of the hole, the condition of the hole as

regards water, and the angle at which the hole is driven. The rate of drilling also depends on the method used, the type and condition of the drill used, and the skill used in handling the drill.

With a hammer drill, cutting a 1½-in. hole, one man handling the drill and two men striking, the rate of drilling a 6-ft. hole is about as follows:[1]

Granite	7 ft. in 10 hr.
Trap (basalt)	11 ft. in 10 hr.
Limestone	16 ft. in 10 hr.

Churn drilling is more economical than hammer drilling for cutting vertical holes. This is especially true where deep holes are to be cut in soft rock such as shale or sandstone. In soft rock three men can drill about 10 ft. of 1½ in. to 2¾ in. in a 10-hr. day. This output will be reduced as the hardness of the rock increases, and be about 7 ft. for granite and 4 ft. for porphyry.

4b. Machine Drilling.—Power drills are generally classified as to the kind of power used in their operation. The best known forms are as follows:

1. Steam Drill
   (*a*) Piston
   (*b*) Cable
   (*c*) Rotary
2. Compressed Air Drill
   (*a*) Piston
   (*b*) Hammer
   (*c*) Rotary
3. Electric Drill
   (*a*) Piston
   (*b*) Hammer

There are many modifications of the above types where an electric motor is used to drive the air compressor for an air drill, or a gas engine is utilized to operate the piston of a so-called piston or percussive drill.

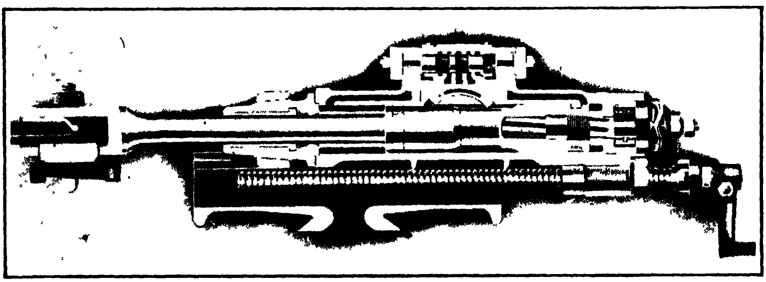
The types of machine drills in general use in rock excavation are the piston drill, the hammer drill, and the cable drill.

The piston or percussive drill consists of a piston, to which the drill rod is attached, and which reciprocates within a cylinder. The motion of the rod and piston is controlled by a valve or

[1] H. P. Gillette. "Handbook of Rock Excavation."

series of valves. The rotation of the drill is provided by means of a rifle-bar, pawls, and ratchet-wheels. The drill is fed forward by a hand-operated crank at the end of a feed-screw. For large drills and deep holes, an automatic feed is provided by means of a tappet which enters the front end of the cylinder and is struck by the piston, thus turning the ratchet at the rear of the cylinder. Fig. 23 shows the principal parts of an improved form of percussive drill.

The hammer drill may be operated by steam, air, electricity, or gasoline, but the two former forms of power are ordinarily



(*Courtesy of Ingersoll-Rand Co.*)

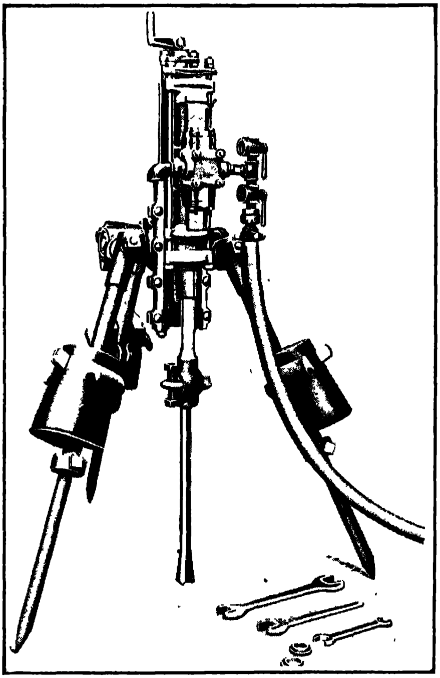
FIG. 23.—Sectional view of power drill.

used. In this form of drill, the drill steel is not attached to the piston as in the piston or percussive type, but is separated and struck with a hammer action by a piston, anvil block, or striking pin. The power-hammer drill is similar to the hand-hammer drill, while the percussive drill resembles in action the hand-operated churn drill. The hammer drill consists of a casing, which contains the striking piston (acting in a cylinder under a pressure of 100 to 125 lb. per sq. in.), the steel or drill holder, the rotating mechanism, and the feed appliance. Rotation is provided for by a rifle bar, one end of which enters one end of the striking piston, while the head has pawls which engage the ratched ring and provide for positive rotation of the piston. The necessary rapid action of the hammer drill is provided by having large parts and a quick acting valve. The latter is often of the butterfly type.

Power drills may be held in the hand, mounted on a tripod, a column, a drill carriage, or frame. The tripod is the customary method of support in open excavation, while the column is used in tunneling and other forms of support in quarrying and mining.

Fig. 24 shows a hammer drill, mounted on a tripod, in operation. Hand-power hammers have come into general use for drilling shallow holes, for block-holing, trimming slopes, etc. The Jack-hammer is a well-known form of air-operated drill used in open rock excavation (see Fig. 25).

The pulverized rock is removed from the hole during drilling by jets of steam, air, or water. The best method is a combined



(Courtesy of Ingersoll-Rand Co.)

Fig. 24.—Tripod mounted drill.

stream of air and water, which serves to blow out the chips, dissolve the dust, and cool the bit.

During the last quarter of a century, the demand for a type of drill, which will be of sufficient size and depth for blast-hole work in deep excavation, has led to the development of the cable or blast-hole drill.

Drills may be tractive or non-tractive. In the latter type the drill is self contained on a stationary or movable platform or frame, but cannot move under its own power. In the first

type, the drill is mounted on a four-wheel truck which is propelled by a chain and sprocket connection to the operating engine.

The well or churn drill generally consists of a rectangular timber framework which supports the machinery and is carried on a fixed rear truck and a pivoted front truck. Fig. 26 shows two churn drills operating on the top of the ledge of a cement quarry.

On the front end of the frame is placed the vertical guides which are provided with a sheave at the top for the operating cable.

Fig. 25.—Jackhammer drills.

Between the axles and on the vertical part of the truck is placed the machinery which is operated by the power equipment, located on the rear end of the platform. The power equipment is generally a vertical engine operated by steam, gasoline, or compressed air. Where electric power is available at a reasonable cost of about $1\frac{1}{2}$ to 2 cts. per kilowatt hour, this form of power has many advantages over other forms. It is economical in first cost, the equipment consisting of an electric motor, which is compact, clean, and easy to operate. The gas engine is also an efficient and economical form of power generator and can be used to great advantage where coal is expensive and requires

long hauls, and electric-power current is not available. Compressed air is clean and easy to use but requires an expensive initial plant installation and the maintenance of a complicated system of piping and appurtenances.

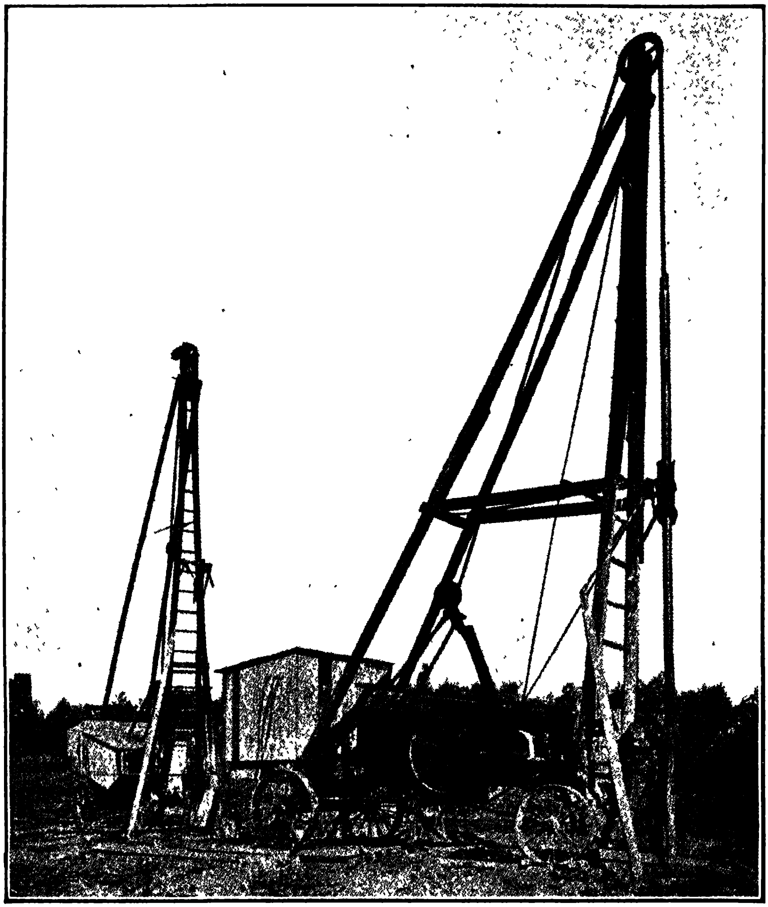


FIG. 26.—Churn drills on quarry work.

Steam may be supplied from a nearby plant through pipes to operate the engine, but generally the drill is located at such a distance from the power plant that this is impracticable and the steam is supplied from a vertical type of boiler located on the truck platform near the engine. Where coal is cheap and good

water is available, this form of power is economical. However for ease of operation, cleanliness, compactness of plant, economy of repairs, and mechanical efficiency, the use of electric generators or internal combustion engines is preferable, where local conditions are favorable. Fig. 26 shows a cable drill operated by an electric motor.

Scope of Work.—The hammer drill is the most efficient type of drill to use for light and rapid work where a portable, quick-action machine can be employed. Holes up to 10 or 12 ft. in depth are put down with drills held in the hands of the operator, although for general use the drills are mounted.

The piston or percussive drill is adapted to heavy work where greater weight and higher pressures are required than obtain with the hammer type of drill. The piston drill for ordinary work is mounted on a tripod, but for excavation of large areas, it is often mounted on a truck and may be auto-tractive. The hand drills use drill steel of from 5/8 to 1 in. in diameter, the mounted drills use steels of from 1 to 2 in. in diameter, while the heavier, auto-tractive machines use drills from 2 to 4 in. in diameter.

The rate of drilling in rock depends on many factors, such as size of drill, type and kind of machine, character and condition of rock, skill of operator, etc. Studies made by a well-known authority[1] suggest the following rate of drilling with 3 1/8-in. machines, using air or steam at 70-lb. pressure, starting bit 2 3/4 in. and finishing with a 1 1/2-in. bit:

	Time to Drill 1 Ft.
Soft sandstones, limestones and shales........	3 minutes
Medium sandstones, limestones and shales....	4 minutes
Hard granites, sandstone and limestones......	5 minutes
Very hard granites, traps, etc................	6–8 minutes
Soft rocks that sludge rapidly................	8–10 minutes

For deep foundation or pit excavation, where open face blasting may be done, the cable drill is the best type, as it will drill large size holes to any depth at a minimum of cost. The relative advantages of the cable and the smaller types (hammer and piston) of drills for deep, heavy rock excavation are as follows:

First.—The large drills can be operated to full depth of excavation and thus only one set of holes are made and used. The expense and difficulties of working one or more intermediate benches are eliminated. Bench gangs are not needed and only one pit gang on the floor is necessary to handle the excavated stone.

[1] H. P. Gillette, "Handbook of Rock Excavation."

Second.—The cost of drilling per ton of rock excavated is less with the large holes than with the small holes. The large holes are spaced farther apart and are fewer in number for the same volume of rock. For example, let us assume that on the same limestone bench, we have a 2¼-in. tripod drill and 5½-in. blast-hole drill of the well-driller type. The average output for the small drill will be 100 ft. of 2½-in. hole and for the large drill will be 60 ft. of 5¾-in. hole. Each square inch of hole will displace about 6 sq. ft. of stone and the 2½-in. hole will therefore take care of 29.46 sq. ft. and the 5¾-in. hole, 175.82 sq. ft. of rock. This means that there will be required $\frac{175.82}{29.46}$, or six times as many small holes as large holes in order to remove the same quantity of rock. The small drill penetrates 100 ft. while the large drill goes 60 ft. into the rock. Thus, it will require four small drills to perform the equivalent work of one large drill. The cost of operation of the large drill is about 50 per cent greater than for the small and $\frac{4}{1.5}$ equals 2.5 approximately. Thus, we may state that the cost of drilling with tripod drills is about 2½ times as much as with blast-hole drills of the well-driller type. This rule applies to limestone drilling on a clean level bench and under average working conditions.

Third.—The large holes are easier and cheaper to load and tamp. By using larger masses of explosive spaced farther apart, its efficiency is increased and a saving in cost effected.

Fourth.—The small holes drilled by a tripod drill generally diminish in size toward the bottom while the large holes are of the same size for their full length. This allows the placing of a maximum amount of explosive at the bottom of the hole, where it will be most effective in breaking down the adjacent rock and leaving a clean floor.

Fifth.—The rock in a ledge is rarely of uniform hardness. Seams and cavities occur at intervals and these must be considered in order to secure the greatest efficiency and economy in the drilling and blasting. Large holes spaced at greater distances can be used to better advantage in this respect than smaller holes spaced nearer together.

The rate of drilling for a cable drill placing a 5⅝-in. hole, will vary with conditions. During a 10-hr. day, an average output should run from 60 ft. in soft sandstone or limestone to 15 and 25 ft. in trap and granite.

4c. Explosives.—Explosives are substances used to generate power for the breaking-up of masses of rock. The power is generated by an explosion, which is the chemical action between the elements of the explosive substance. This action takes place at a high temperature and results in the generation of a large volume of gas. The sudden expansion of the gas, which is confined in a relatively small space, produces the explosive force which ruptures or breaks up the rock.

There are two classes of explosives: black powder, and dynamite and nitroglycerin.

Black powder is a slow burning compound of a low explosive order. It consists of 70 to 75 per cent saltpeter, 10 to 15 per cent charcoal, and 10 to 15 per cent sulphur. In the inferior grades of black powder, the saltpeter (potassium nitrate) is often replaced by sodium nitrate, which deteriorates upon exposure to the moisture in the air. Hence, great care should be taken to keep the powder in an air-tight container and stored in a dry place. Powder comes in metal containers or kegs of about 25-lb. capacity. The powder is classified in grades depending on the size of the grain, which varies from about $\frac{1}{16}$ to $\frac{1}{2}$ in. in diameter. The rate of combustion varies with the size of grain, and thus a fine grained powder is termed a *quick* powder, while a course grained powder is called a *slow* powder. Powder grains should be uniform in size for any grade, be free from dust, and have no sharp edges or corners.

Black powder, in blasting rock, is poured through a funnel into the hole or holes which have been previously drilled as described in Arts. **4a** and **4b**. In horizontal holes, the powder is placed by a long-handled scoop, or shoved into the holes in paper bags. In wet holes, the powder should be placed in cylindrical paper containers or *cartridges*, which are coated with paraffine to ensure waterproofness. All holes subject to seepage should be cleaned out and the water removed as far as practicable, before loading. The powder is ignited by a cap or fuse, the lower end of which is buried in it. After the hole is loaded, clay or sand should be placed above the charge in layers well compacted by tamping with a wooden bar or rod. When powder is fired by electricity, a paper cap containing powder is placed in the charge and ignites the latter by a small flame.

Dynamite and nitroglycerin are rapid explosives of a high power, and are composed of an absorbent substance termed *dope*, and nitroglycerin. The *dope* is called *inactive* if an inactive substance such as porous earth or wood pulp is used, and *active* if the absorbent material is gunpowder. The grades of *straight* dynamite are determined by the amounts, expressed in percentage by weight, of nitroglycerin contained in the explosives. Thus, if the weight of nitroglycerin is 60 per cent of the total weight of the compound (with inactive dope), the explosive is termed a 60 per cent dynamite. Dynamites with an active base

are rated by comparison with a standard, inactive dope, brand. Atlas powder is one of the best known brands of *straight* dynamite.

The classes of dynamite in general use are: the straight nitroglycerin dynamite, the low freezing dynamite, the ammonia dynamite, and the gelatin dynamite. The low freezing dynamite will not generally freeze at temperatures above 32 deg. F. The ammonia dynamite has a small percentage of ammonium nitrate, which increases the explosive power. Gelatin dynamite contains a very small percentage of nitrocellulose, which gives a higher explosive power and renders the material impervious to water.

Dynamite comes in paper cartridges, 7/8, 1, 1 1/4, to 2 in. in diameter, and varying in length from 6 to 16 in. The common size is 1 1/4 × 8 in. and is shipped in wooden boxes holding 25 or 50 lb.

The cartridges, which should be slightly smaller than the drill holes, are slit lengthwise with a knife and pressed well into the hole, successively, by a wooden rammer. In the last cartridge is placed a fuse cap or electric detonator. This is done by inserting the cap into a hole bored in one end of the cartridge with a rounded stick. The fuse cap is allowed to extend about 1/8 in. beyond the end of the cartridge, and the ends of the paper are tied tightly around the fuse, and enclosing the cap. The primer (cartridge and cap) is lowered into the hole but not by means of the fuse. Moist clay or sand is tramped in well compacted layers above the charge.

When the charge of dynamite is to be fired by electricity, the primer is loaded with an electric detonator, which may be inserted in the end or side. Fuse wires extend from the primer to a firing machine or blasting battery, which is a hand-operated dynamo consisting principally of two electro-magnets and an armature The pushing or pulling of a handle generates an electric current, which transmitted to the detonator through the wires, explodes the charge.

Dynamite is a dangerous material and should be handled with care. It should never be stored with caps, and when frozen should be thawed out by the application of indirect heat in a space or chamber adjacent to a fire or other source of heat.

Field of Use.—The kind or strength of explosive to use in any particular case depends on several factors, such as the nature and

condition of the material to be blasted, the amount of breaking up required, the size and spacing of the drill holes, the nature of the excavation as bench, open cut, or tunneling, etc.

Black powder and Judson powders are best adapted to dry excavation where a slow, heaving action is desired. This class of explosive is also efficient in the blasting out of shale and other similar soft and laminated materials.

The straight dynamites are generally the best to use in simple, open-cut work, where a quick, positive, shattering effect is desired. Gelatine dynamites are best adapted for wet work. In order to secure maximum efficiency from this class of explosive, the strongest detonator should be used. The ammonia dynamites are slower than straight dynamites, are not so sensitive, and should be used where too great a shattering effect is not desired.

Scope of Work.—The method of blasting to be used depends on many factors, and in foundation or pit excavation is generally done in sections or by the so-called *bench* method.

No rule can be given as to the spacing of the holes, as this depends largely upon the nature and condition of the rock, the required size of fragments, kind of explosive used, etc. In general practice, the holes are placed back from the face a distance about equal to their depth. The holes can often be spaced a distance apart considerably greater than their depth in stratified rock. When stratified rock has a dip, the spacing of the holes parallel and at right angles to the strike should depend on the kind of explosive used, the character of rock, the size and numbers of seams or fissures, and the method of loading or placing the charges. As a general rule, holes should never be more than 20 ft. apart, and in hard rock, it is well to try out spacings of from 10 to 15 ft.

The amount of explosive to place in a charge depends on so many variable elements that no absolute rule can be given. It is common practice to fill the larger size holes on heavy work to one-half their depth, and to bring the charge up to not less than 30 ft. from the surface. In hard material, the holes are often loaded up to within 20 ft. of the surface. In deep holes, two or more charges may be placed so as to secure greatest efficiency with economy in the use of the explosive. In ordinary rock 40 per cent dynamite will give satisfactory results, but in hard rock, a combination of 60 per cent and 40 per cent dynamite is advisable—the 60 per cent placed in the bottom of the hole, and

TABLE 6.—CUBIC YARDS OF ROCK REMOVED PER FOOT OF HOLE

Distance holes are set back from face (feet)	Distance apart of holes (feet)							
	5	6	7	8	9	10	11	12
	0.92	1.11	1.3	1.49	1.66	1.85	2.04	2.22
	1.11	1.33	1.55	1.77	2.0	2.22	2.44	2.65
	1.3	1.55	1.81	2.0	2.33	2.7	2.85	3.11
	1.49	1.77	2.0	2.37	2.65	2.96	3.26	3.55
9	1.66	2.0	2.33	2.65	3.0	3.33	3.66	4.0
10	1.85	2.22	2.7	2.96	3.33	3.7	4.1	4.44
11	....	....	....	3.26	3.66	4.1	4.48	4.88
12	....	....	....	....	4.0	4.44	4.88	5.33
13	....	....	....	....	....	4.81	5.3	5.77
14	....	....	....	....	....	5.18	5.7	6.22
15	....	....	....	....	....	5.55	6.11	6.66
16	...	..	....	....	...	..	....	7.11
17	....	.	....	....	...	..	...	7.55
18	..	...	...	..	...	...	....	8.0

Distance holes are set back from face (feet)	Distance apart of holes (feet)							
	13	14	15	16	17	18	19	20
10	4.81	5.18	5.55	5.92				
11	5.3	5.7	6.11	6.52				
12	5.77	6.22	6.66	7.11				
13	6.26	6.74	7.22	7.70				
14	6 74	7.26	7.77	8.30				
15	7.22	7.77	8.33	8.88	9.44	10.0	10.55	11.11
16	7.70	8.30	8.88	9.48	10.07	10.66	11.3	11.85
17	8.18	8.81	9.44	10.07	10.70	11.33	11.96	12.59
18	8.66	9.33	10.0	10.66	11.33	12.0	12.66	13.33
19	9.15	9.85	10.55	11.3	11.96	12.66	13.37	14.07
20	9.63	10.37	11.11	11.85	12.59	13.33	14.07	14.81
21	....	.....	11.66	12.44	13.22	14.37	14.77	15.55
22	....	.....	12.22	13.03	13.85	14.66	15.48	16.30
23	....	.....	12.78	13.63	14.48	15.33	16.18	17.03
24	....	.....	13.33	14.22	15.11	16.0	16.88	17.77
25	....	....	13.88	14.81	15.74	16.66	17.60	18.51
26	...	.....	14.44	15.74	16.37	17.33	18.30	19.26
27	..	.....	15.0	16.15	17.0	18.0	19.0	20.0
28	....	.....	15.55	16.6	17.63	18.52	19.7	20.74
29	....	.....	16.1	17.18	18.26	19.33	20.4	21.48
30	...	.....	16.66	17.77	18.88	20.0	21.1	22.22

To reduce to tons:
- For limestones..................... multiply by 2.27
- For traps, syenites, etc............ . multiply by 2.52
- For granites............................ multiply by 2.3
- For shale .... multiply by 2.18
- For glass sand or gravel............. multiply by 1.55

the 40 per cent on top. It is well to space caps or detonators about every 25 ft. in deep holes.

Usually the holes are exploded in lines equidistant from the face. The fractured material is then removed before the next blast is made. In very hard material, good results are secured by staggering the holes. In some cases, a second line of holes is shot down before the blasted material from the first line of holes is removed. This is the so-called *buffer* method of blasting and is especially adapted to loose stratified material that can be easily handled by an excavator in the cut.

Table 6 gives the number of cubic yards of rock removed per foot of hole at different spacings.

Table 7 gives the number of pounds per linear foot of hole of various types of explosives which can be loaded into holes of different diameters, assuming the cartridges are slit and well tamped.

TABLE 7.—POUNDS PER LINEAR FOOT OF HOLE OF VARIOUS TYPES OF EXPLOSIVES

Dia. of hole (inches)	Gelatine Dynamite	Straight N. G. Dynamite	R. C. Str. or L. F. Dynamite	R. C. X. or L. F. Amer. Dynamite	Ex. or Amer. Dynamite	Arctic or N. S. Explosives	Judson Railroad Powder	Black Blasting Powder
3	4.25	3.75	3.68	3.60	3.72	3.25	3.25	3.00
3½	5.68	5.10	5.0	4.89	5.07	4.42	4.42	4.08
4	7.55	6.60	6.53	6.33	6.62	5.72	5.72	5.28
4½	9.35	8.40	8.38	8.06	8 38	7.28	7.28	6.72
5	11.80	10.50	10.2	10.08	10.35	9.10	9.10	8.40
5⅝	14.94	13.2	12.8	12.53	13.0	11.3	11.3	10.44
6	17 0	15.0	14.7	14.4	14.9	13.0	13.0	12.0
6½	19.5	17.5	17.25	16.8	17.49	15.2	15.2	14.0
7	23.1	20 4	20.0	19.6	20.28	17.7	17.7	16.3
8	30 2	26.7	26.13	25.6	26.49	23 1	23.1	21.4

4*d*. Rock Breaking.—The blasted rock as it lies on the floor of the excavation may vary in size from a pea to fragments several feet in their least dimension. In order that the larger pieces of rock may be loaded into skips or buckets by hand or by some form of excavating machinery, it often becomes necessary to break them up into smaller fragments. The following methods are used:

(*a*) Dropping of heavy weights
(*b*) Use of sledge hammers
(*c*) Block holing
(*d*) Mud capping
(*e*) Undermining

(*a*) The method of dropping heavy weights from a considerable height is only practicable when a derrick, locomotive crane or cableway is available. The weight used is ordinarily a block of cast iron weighing about one ton. The height of the drop should be from 15 to 30 ft., and the weight released suddenly by a trip of a friction-drum engine. This method is applicable in a restricted space where a derrick is available.

(*b*) The sledge hammer is one of the oldest and simplest methods of breaking up rock fragments. The sledge used should be of such proportions that a man of average strength can wield it efficiently. A lighter sledge of about 12-lb. weight, used with rapid blows, is much more effective than one weighing 16 lb. and used with slow strokes.

This method, though crude, is efficient for the breaking up of fragments to about 1 cu. yd. in volume, and for sedimentary and stratified rock. Every pit gang should be supplied with sledges for the breaking up of the rock fragments which are too small to be economically broken up by block holing or mud capping, and yet are too large and unwieldy to be handled by the power excavator.

(*c*) *Block holing* is the simple application of the ordinary method of drilling and blasting to the breaking up of large rock fragments. The most efficient method is to drill a hole from a few inches to 2 ft. in depth and place the center charge of explosive in the hole. But, the more common and quicker way is to drill a shallow hole and place a small part of the explosive on the rock above the hole and cover the entire charge with mud.

This method is the most effective one for the breaking up of rock larger than 1 cu. yd. in volume, and is in common use in open excavation work.

(*d*) *Mud capping* is probably the most popular method of breaking up rock, especially with the average pit or blasting foreman. The placing of the explosive directly on the surface of the rock and covering it with a mud blanket is a simple but an expensive and uneconomical process. The resulting efficiency of the explosive is very low.

This method should not be used except when it is necessary to break up rapidly huge pieces of rock, which are delaying the operation of the excavators.

(*e*) The method of undermining is little used and not generally known. It resembles *mud capping,* but differs essentially in

that the explosive is under rather than on top of the rock. Where large masses of rock are piled up together, the charge of explosive can be placed on the upper surface of the lower fragments and thus have a bed. The same results can be attained by this method as by *mud capping* with the use of about one-half the amount of explosive.

5. Subaqueous Rock Excavation.—The earliest method of subaqueous rock excavation consisted of the use of explosives, which were lowered to the surface of the rock. This method proved to be uncertain and unsuccessful, especially in the case of ledges of hard rock. Large boulders and projecting rock can be broken up satisfactorily by this method. In some cases, a drop bar was used to drill holes, into which charges were placed in the regular way. This method was slow and expensive. The unwatering of the surface of the rock by constructing cofferdams or by lowering caissons and pumping out the water is often used in bridge pier work.

In England and Germany the drilling and removal of rock in shallow water has been done by means of bottomless boxes, called *diving bells*. This method is limited in scope as it can only be used in quiet water and over small areas.

The two general methods of subaqueous rock breaking are as follows: (1) By the use of a heavy pointed bar or chisel such as the Lobnitz Rock Cutter, and (2) by some form of drill boat.

5a. Lobnitz Rock Cutter.—The Lobnitz Rock Cutter consists of a heavy chisel of iron or steel weighing from 4 to 20 tons, and provided with a hardened steel cutting point. The cutter or cutters are mounted on a hull or barge which is rigidly braced by cross frames.

The cutter is operated by a steel cable which is attached to the top of the cutter and then passed over a sheave suspended in an A-frame vertically above the cutter, and thence to the drum of a hoisting engine. Figs. 27*A* and 27*B* give a detail view of a Lobnitz Rock Cutter.

The operation of the cutter is similar to that of a drop-hammer pile driver, the fall varying from 5 to 10 ft. The impact of the falling point will fracture rock from 1 to 3 ft. in depth, depending on the nature and structure of the material.

In Europe where the rock breaker is in general use, the ladder dredges are often equipped with several picks or cutters, located in a well alongside the ladder. The picks or chisels are placed

about 2 ft. apart and can be operated singly or co-ordinately. The buckets of the dredge are provided with heavy steel teeth for the removal of the rock fragments.

5b. Drill Boats.—There are four forms of drill boat in general use:

(*a*) A floating barge, equipped with movable towers on which the drills are mounted.

(*b*) A floating barge, equipped with drilling frames which are arranged to lower the drills to the rock surface.

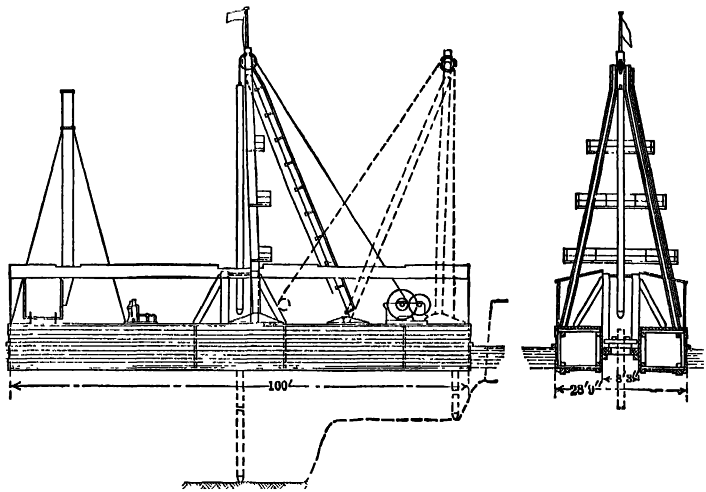


Fig. 27*A*.—Side elevation and cross-section of Lobnitz rock cutter.

(*c*) An adjustable platform which supports the drills and can be raised and lowered.

(*d*) A floating platform or barge, equipped with tripod drills.

The drill boat consists essentially of a barge equipped with a spud at each corner, and carrying one or more power drills. The details of construction depend on the uses to which the boat is to be put, and especially the character of the stream—tidal or non-tidal.

The drilling equipment consists of power-operated drills which are either fixed or mounted on movable towers. The latter may be placed on a track and moved by means of a special engine. The drill may be of the cable or percussive type and is generally operated by steam. Compressed air has proved to be

uneconomical and unsatisfactory. Power for steam-operated drills is supplied by specially constructed engines with a throttle reverse. Fig. 28 shows a detail view of a submarine rock drill, and Fig. 29 a drill frame and engine.

In localities of low tidal range, the drill boat may be raised 4 to 6 ft. on its spuds, and operate the drills satisfactorily. Where high tidal range and wave action obtains, the drills are mounted on frames, which are fastened to heavy timber or

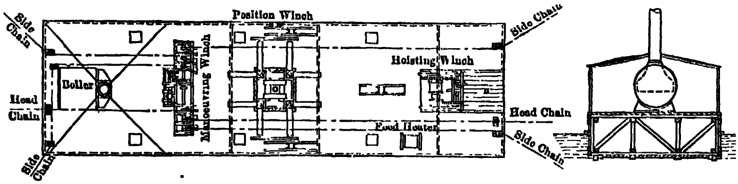
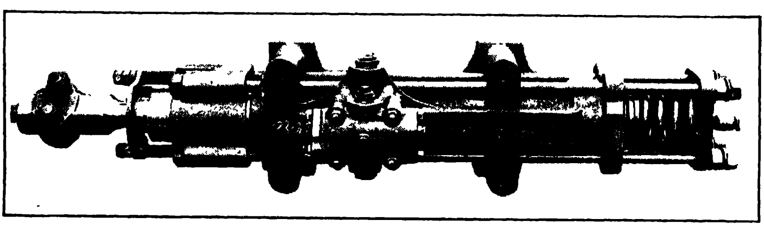


FIG. 27B.—Plan view of Lobnitz rock cutter.

steel spuds. The frames are so mounted that they can be raised and lowered by cables connected to a hoisting engine. Fig. 30 shows a boat equipped with two adjustable drill frames. Where swift currents, high tides, and rough water occur in shallow depth of water, an adjustable platform can be used.



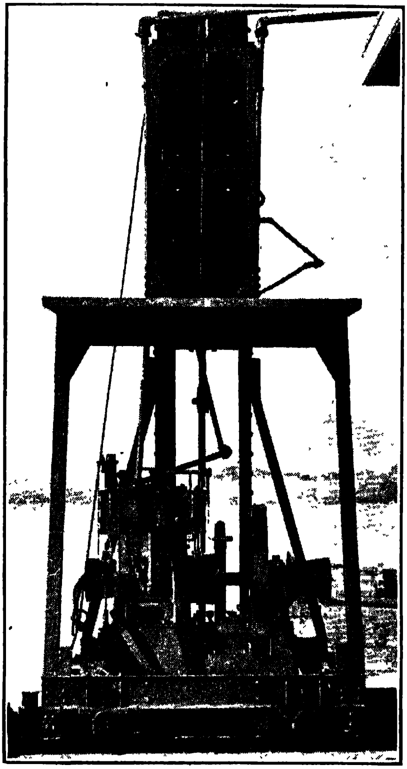
(Courtesy of Ingersoll-Rand Co.)

FIG. 28.—Submarine rock drill.

Flotation is secured by barrels or pontoons, and a spud at each corner is used to support the platform. The drills are generally of the percussive type mounted on tripods or wooden frames, which are arranged to move longitudinally over slots in the platform. The platform and spuds are operated by a hand winch. The power equipment of boiler, hoisting engine, etc. is usually placed on a barge moored adjacent to the platform. In shallow waters where the tidal range is small, as in inland

waters, the boat consists of a floating pontoon or raft which carries one or more tripod drills.

The drills vary from 1½ to 4⅝ in. in diameter, depending on the type and setting of the drill. The holes are spaced 2 to 5 ft. on centers, and are loaded through a tube or pipe which

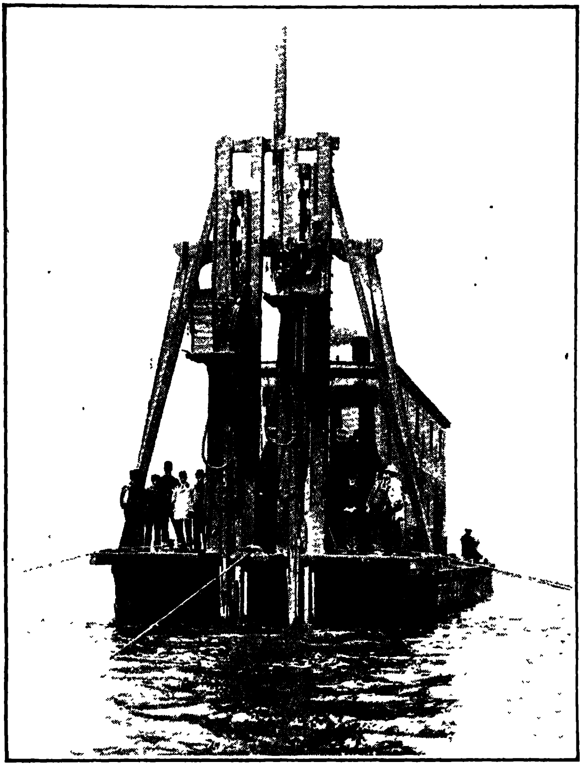


(*Courtesy of Ingersoll-Rand Co.*)
FIG. 29.—Drill frame and engine.

rests on the rock surface. The charge may vary from ½ lb. of 40 per cent dynamite to 1 lb. of 80 per cent dynamite per foot of hole, depending on the nature of the rock. The charge is exploded by an electric firing machine.

The blasted rock is removed by a dipper or a ladder dredge supplemented at times by derrick scows for the removal of large projecting pieces of rock, which were too large to be handled by the dredge.

Field of Work.—The excavation of rock for piers, docks, sea-walls, and similar structures, the foundations of which lie under water, involves the breaking up of the rock and its subsequent removal. The method of breaking up depends on the nature



(*Courtesy of Ingersoll-Rand Co.*)

Fig. 30.—Drill boat with adjustable drill frames.

of the rock. The rock cutter works most efficiently in shallow layers of soft, stratified rock, such as shale and sandstone. Hard rock as trap, gneiss, or granite, especially in layers over 3 ft. in thickness, can be handled more economically by the drill boat.

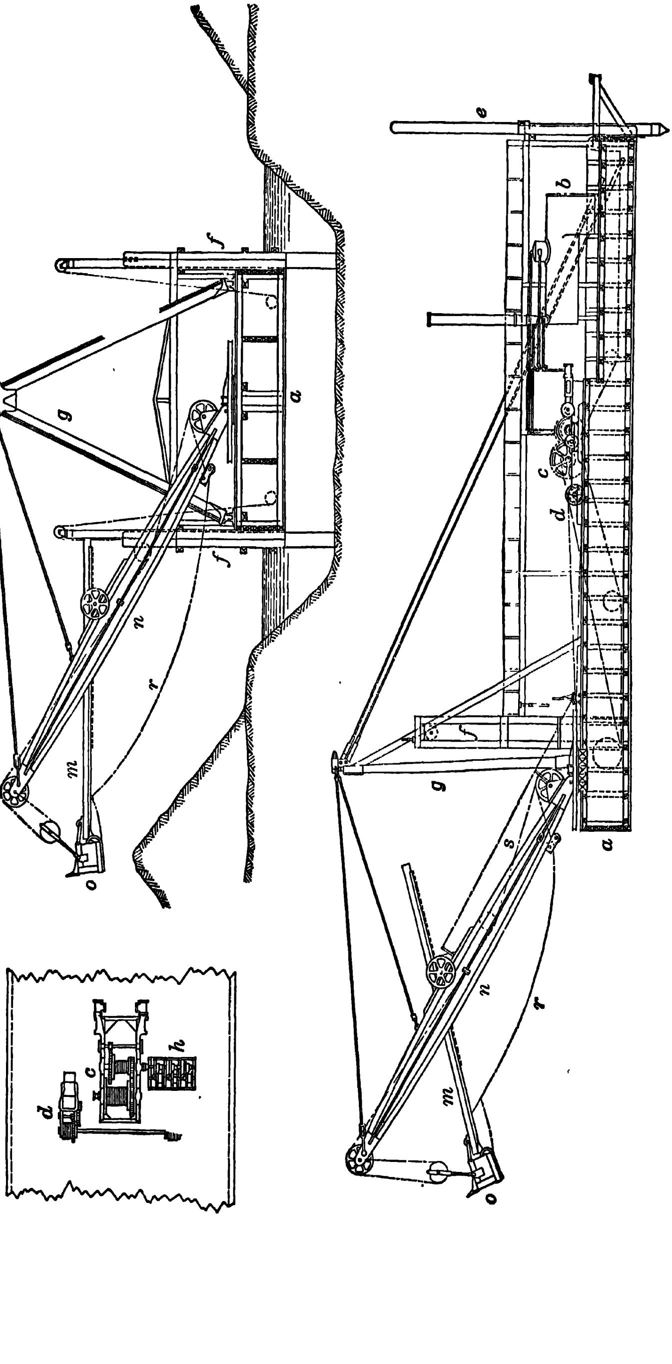
The output of the rock breakers will vary with the local conditions, such as depth of water, size of cutter, nature of material, etc. In ordinary rock—such as sandstone and limestone—a chisel weighing about 10 tons will deliver about 150 blows per hr. and break up about 10 cu. yd. On the Panama

Canal, during 1910–11, a 19-ton ram, with a drop of 5 to 14 ft., broke up about 35,000 cu. yd. of rock in 3535 working hours, or at the rate of about 10 cu. yd. per hr.

The output of a drill boat is dependent on so many variable factors of size and number of drills, local conditions of water, weather and tide, nature of material, etc., that it is impractical to give any definite rules as to capacity of this type of rock drill. At Oswego, N. Y. in 1894, 5-in. diameter holes, 5 ft. on centers, were drilled in graywacke sandstone to a depth of about 5 ft., at the average rate of one hole an hour. In 1905, five drill boats at Buffalo, N. Y. averaged 86 ft. per drill per 24-hr. day, in the drilling of holes spaced 4 by 5 ft. and 23 ft. deep through flinty limestone rock. A drill boat equipped with four, hydraulic operated, 5½-in. drills operated in 1911 on the Detroit River, and averaged over four months, 390 ft. of holes per 11-hr. shift. The holes were spaced about 5-ft. centers and varied from 11 to 16 ft. in depth.

6. Wet or Subaqueous Excavation.—The excavation of earth and rock under water as in the preparation of the foundations for docks, piers, sea walls,—breakwaters, etc.—can best be done by some type of floating excavator, if the work covers sufficient area and is of a sufficient magnitude to warrant it. The excavation of bridge piers, pits, and work of a similar type, where the area to be excavated is very limited and the depth is great, is usually done by some special form or method, such as the cofferdam, the pneumatic caisson, and dredging through wells. These methods will be discussed in other chapters of this volume. It is the purpose of this article to deal with the construction, operation, and use of the common type of floating excavator; the dipper or bucket dredge, the grab-bucket dredge, the hydraulic dredge, and the ladder dredge.

6*a*. Dipper Dredges.—Dipper dredges are built in three different types, depending on the use to which they are to be put: (1) the narrow hull dredge with side or bank spuds for ditch excavation, (2) the narrow hull dredge with side floats for channel excavation and maintenance, and (3) the heavier, broad hull dredges with vertical spuds for river and harbor work. The general features and essential parts of all three classes are the same and consist of the hull, the power equipment of hoisting engines and swinging engines, and the excavating equipment of A-frame, spuds, boom, and dipper or bucket.



(*Courtesy of Marion Steam Shovel Co.*)

FIG. 31.—Diagram of floating dipper dredge. *a*, Hull; *b*, boiler; *c*, hoisting or main engine; *d*, swinging engine; *e*, rear spud; *f*, side spuds; *g*, A-frame; *m*, dipper handle; *n*, boom; *o*, dipper; *r*, latch rope; *s*, hoist line.

The proportions of a dredge depend on its working capacity and the hull, power equipment, and boom must all be designed co-ordinately. The dimensions and capacities of dipper dredges with vertical spuds and dipper capacities from ¾ to 4 yd. are given in Table 8.

The essential parts of a typical dredge are shown in Fig. 31. The dipper dredge is operated by a steam plant, and a feed water heater and purification system to provide suitable boiler water. In the larger sizes of dredge, the swinging engine is independent and operates the boom by a fixed turntable above the deck. The spuds are generally operated by separate engines, one at each spud.

The method of operation of a dipper dredge is similar to that of a steam shovel, and the crew consists of an engineer, a cranesman, a fireman, and from two to four laborers for each shift.

Field of Use.—The dipper dredge is one of the most universal and adaptable excavators for subaqueous work. In the preparation of a foundation for a dock or pier, it can pull stumps or piles, remove boulders, bridges, and other obstructions, drive piling, build temporary walls or dams, and excavate all kinds of soil from silt to loose rock. The great thrusting and prying power of the dredge makes it an especially efficient machine in the removal of the tougher materials which cannot be handled by the other types of floating dredge.

The output for a dipper dredge varies with the conditions, such as kind and condition of material, size and efficiency of dredge, climate, character and depth of excavation, etc A 2-yd. dredge will excavate from 600 cu. yd. of hard clay to 1000 cu. yd. of soft clay, under average working conditions, in a 10-hr. shift.

In the excavation of the shoals, preliminary to the construction of a large Boston storage depot during 1918 and 1919, eight dipper dredges operated. These dredges varied in size and capacity from 2½ to 15 yd. and worked over different periods. The material excavated was silt, sand, gravel, clay, and some boulders and rock. Following is a brief resume of the operations:

A 2½-yd. dredge, rated capacity of 1400 cu. yd. in a 10-hr. shift, excavated 330,923 cu. yd. in 2726 hr. The average rate of excavation was 139 cu. yd. per working hour.

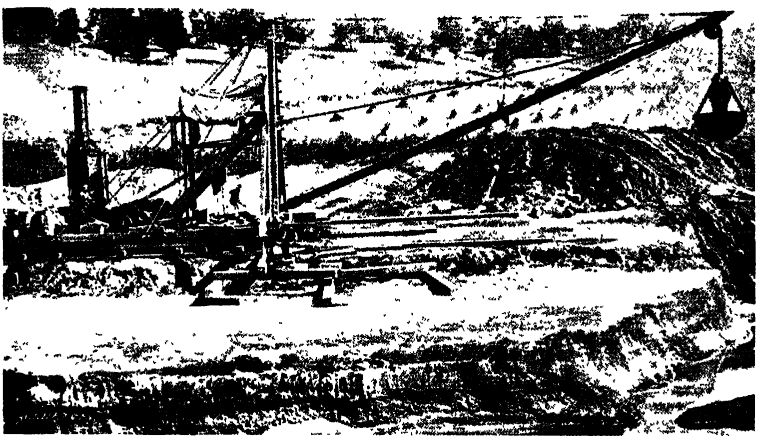
A 5-yd. dredge, rated capacity of 1000 cu. yd. per 10-hr. shift, excavated 259,818 cu. yd. in 3554 hr. The average rate of excavation was 1260 cu. yd. per hour worked.

TABLE 8.—DITCHING DREDGES WITH VERTICAL SPUDS
Dippers—¾ to 4 yd.
Booms—30 to 100 ft. long

Size of dipper (cu. yd.)	Length of boom (ft.)	Height dump above water (ft.)	Depth dig below water (ft.)	Center hull to center dump (ft.)	Size of hull (ft.)	Hoisting engines (in.)	Swinging engines (in.)	Capacity (cu. yd.)	M-ft. lumber for hull (B. M.)
4	100	36–40	28	85–100	120×50×8½	2–12 ×16	2–9 ×9	1,500–3,000	195
4	90	32–36	26	75–90	120×46×8½	1–12 ×16	2–9 ×9	1,500–3,000	180
2½	80	30–34	24	67–80	100×42×8	2–10½×12	2–8 ×8	1,000–2,000	135
2½	75	28–32	23	63–75	100×40×8	2–10½×12	2–8 ×8	1,000–2,000	128
2½	70	26–30	22	60–70	100×38×8	2–10½×12	2–8 ×8	1,000–2,000	122
2½	65	24–28	21	55–65	90×36×7½	2–10½×12	2–8 ×8	1,000–2,000	98
2½	60	22–26	20	51–60	90×34×7½	2–10½×12	2–8 ×8	1,000–2,000	92
2½	55	20–24	18	47–55	90×32×7½	2–10½×12	2–8 ×8	1,000–2,000	87
2½	50	18–22	16	43–50	90×30×7½	2–10½×12	2–8 ×8	1,000–2,000	81
2½	45	16–20	14	39–45	90×28×7½	2–10½×12	2–8 ×8	1,000–2,000	76
2	65	24–28	21	55–65	85×36×7	2– 9 ×11	2–7 ×8	800–1,600	86
2	60	22–26	20	51–60	85×34×7	2– 9 ×11	2–7 ×8	800–1,600	81
2	55	20–24	18	47–55	80×32×7	2– 9 ×11	2–7 ×8	800–1,600	76
2	50	18–22	16	43–50	80×30×7	2– 9 ×11	2–7 ×8	800–1,600	68
2	45	16–20	14	39–45	80×28×7	2– 9 ×11	2–7 ×8	800–1,600	63
2	40	15–18	12	35–40	80×26×7	2– 9 ×11	2–7 ×8	800–1,600	59
1½	55	20–24	18	47–55	75×32×6½	2– 8 ×10	2–6 ×7	600–1,200	63
1½	50	18–22	16	43–50	75×30×6½	2– 8 ×10	2–6 ×7	600–1,200	59
1½	45	16–20	14	39–45	70×28×6½	2– 8 ×10	2–6 ×7	600–1,200	51
1½	40	15–18	12	35–40	70×26×6½	2– 8 ×10	2–6 ×7	600–1,200	48
1½	35	14–16	10	30–35	70×24×6½	2– 8 ×10	2–6 ×7	600–1,200	44
1¼	45	16–20	14	39–45	65×28×6	2– 8 × 8	2–5½×6	500–1,000	44
1¼	40	15–18	12	35–40	65×26×6	2– 8 × 8	2–5½×6	500–1,000	41
1¼	35	14–16	10	30–35	65×24×6	2– 8 × 8	2–5½×6	500–1,000	38
1	40	15–18	12	35–40	60×24×5½	2– 7 × 8	Friction	400– 800	32
1	35	14–16	10	30–35	60×22×5½	2– 7 × 8	Friction	400– 800	29
1	32	13–15	9	27–32	60×20×5½	2– 7 × 8	Friction	400– 800	27
¾	35	14–16	10	30–35	55×22×5	2– 6½ × 8	Friction	300– 600	25
¾	32	13–15	9	27–32	55×20×5	2– 6½ × 8	Friction	300– 600	22
¾	30	12–14	8	25–30	55×18×5	2– 6½ × 8	Friction	300– 600	20

A 15-yd. dredge, rated capacity 2000 cu. yd. per 10-hr. shift, excavated 840,852 cu. yd. in 4502 hr. The average rate of excavation was 269 cu. yd. per hour worked.

6b. Grab-bucket Dredges.—The grab-bucket or grapple dredge is very similar to the dipper dredge—a hull with steam power equipment, hoisting and swinging engines, and excavating equipment of A-frame, spuds, boom, and dipper. The grab-bucket dredge, however, has a much longer boom than the dipper dredge, often up to 120 ft. in length, and also uses the grab or grapple type of bucket such as the clam-shell or orange-peel bucket. These buckets come in rated capacities of from ½



(*Courtesy of Norbom Engineering Co.*)

Fig. 32.—Grab-bucket dredge excavating canal lock.

to 10 yd., loose measurement (see Figs. 20 and 21). The buckets are operated by two chains or cables; one to close the bucket in loading and the other to open it in discharging. In the simplest form of grapple dredge provided with the automatic swing, the movement of the boom from side to side is controlled by the operating lines, and material can be placed only on one side of the machine. The bull-wheel type of dredge provides for the movement of the boom by a *bull wheel* or swinging circle, which is operated independently of the bucket lines.

The grab-bucket dredge may be mounted on skids or rollers and move over the ground surface while carrying on subaqueous excavation. This is similar to the derrick and hoist buckets described in Art. 3e. Fig. 32 shows such a machine equipped

with a $1\frac{1}{4}$-yd. orange-peel bucket excavating the foundation for a canal lock at the rate of 700 cu. yd. per day.

Field of Use.—The grab-bucket dredge, mounted either on skids for land operation or on a barge as a floating excavator, is especially adapted for the excavation of the softer materials, such as silt, muck, and clay. The clam-shell bucket equipped with teeth can be utilized for the excavation of the softer and lighter soils, while the orange-peel bucket may be used for the excavation of the harder and denser materials, such as sand, gravel, clay, and loose rock. The orange-peel bucket is especially efficient in the removal of boulders, blasted rock, tree stumps, and old piling from the bed of a stream or channel. The long boom on a grapple dredge is of especial advantage in the excavation of foundations lying under water, but adjacent to land, on which the dredge may operate.

In channel excavation, a grab-bucket dredge equipped with a 2-yd. orange-peel bucket will excavate about 2000 cu. yd. of muck and clay in two 11-hr. shifts. In the construction of levees along the Sacramento and San Joaquin Rivers in California, a dredge with a 150-ft. boom and a 14-yd. bucket averaged 8000 cu. yd. of sand and clay in two 11-hr. shifts.

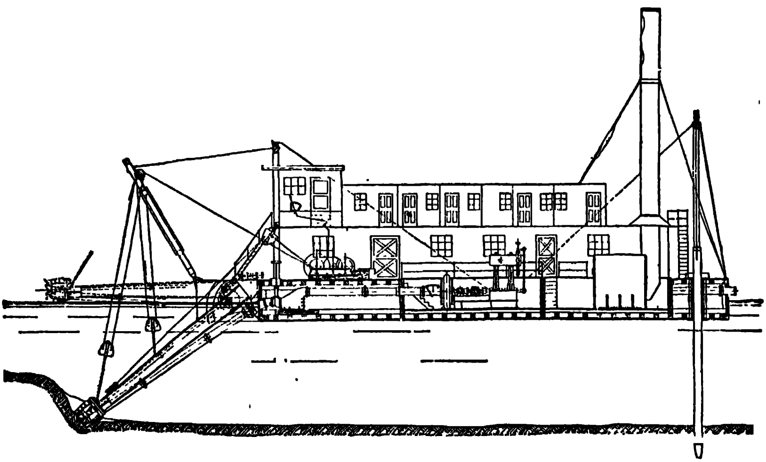
A grab-bucket dredge, equipped with a 7-yd. clam-shell bucket and having a rated capacity of 2000 cu. yd. in 10 hr., removed 415,992 cu. yd. of silt and sand in 3979 hr., during the excavation of the shoals in 1918–19, preliminary to the construction of a large Boston storage depot.

6c. Hydraulic Dredge.—Hydraulic dredges may be classified as to their method of operation and disposition of the excavated material,—namely, the spud dredge, the sea-going dredge, and the Fruhling dredge. The first type, the spud dredge, is especially adapted for channel and foundation excavation where the distance the material must be pumped is not greater than about one-quarter of a mile. There are three different types of spud dredge as follows: (1) Lateral feeding, (2) forward feeding, and (3) radial feeding.

The lateral feeding dredge feeds sideways with the cutter in contact with the bed of the cut or channel. The forward feeding type has the axis of the pump parallel to the axis of the boat and on the center line of the dredge. The suction pipes are provided with vertical flanged joints instead of the radial joints used on the other types of dredge. The radial feed dredge is provided with a cutter which describes an arc of a circle about the spud as a center. The suction pipe is equipped with a universal joint so that it can swing laterally, and also be raised and lowered.

The sea-going and Fruhling dredges are generally large, self-propelled boats with hoppers, and especially constructed to excavate the coarse materials—such as sand and gravel—in deep water, and where storms and rough water may occur.

The essential parts of a hydraulic or suction dredge are the hull, the centrifugal pump, the revolving cutter, and the operating machinery. Attached to the pump is the suction pipe with a



(*Courtesy of Norbom Engineering Co.*)

FIG. 33.—Side view of hydraulic dredge.

flexible movable joint, so that the outer and lower end can be raised or lowered to any desired depth. At the lower end of the suction pipe is placed the mouthpiece, or circular hood. On the periphery of the hood are generally placed a series of knives, which form a revolving cutter. The cutter is revolved by a shaft and gearing, and loosens up the material to be excavated which in dilution is sucked up through the suction pipe. The excavated material, when it passes through the pump, is forced through the discharge pipe, which is supported on pontoons, and discharges into scows or out upon an area to be filled in. Figs. 33 and 34 show the essential parts and construction of a simple spud type of hydraulic dredge.

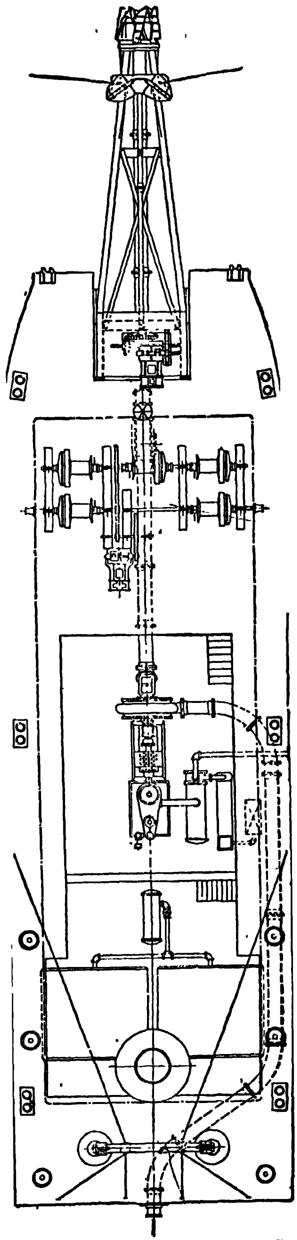
Field of Use.—The hydraulic dredge is the most economical machine for the excavation of large quantities of material under water that is easily removed, and conveyed in dilution considerable distances. Generally the efficiency of this type of excavator is low on account of the high state of dilution of the excavated

material. Hence, in work of magnitude and importance, some method of removing the surplus water in the discharge pipe—as by overflow strainers located in the pipe at intervals—should be used, especially if it is necessary to confine the discharged material in a bank or fill.

A 15-in. dredge operating on two 12-hr. shifts on levee construction on the lower Mississippi River excavated 75,000 cu. yd. of sand and silt per month. An electrically operated dredge with a 20-in. pump, operating on levee construction along the Sacramento River in California, averaged 200,000 cu. yd. of silt and sand per month.

A hydraulic dredge, equipped with a 27-in. discharge pipe, and having a rated capacity in 10 hr. of 3000 cu. yd., excavated 195,847 cu. yd. of silt and sand in 900 hr. in the removal of the shoals preliminary to the construction of a large Boston storage depot.

6d. Ladder Dredge.—The ladder dredge is a type of excavator which has been largely developed abroad, and until the last twenty years, has been little known or used in this country. On account of its high initial cost and relatively limited field of use, the ladder dredge has not and probably never will attain the general use and popularity of the dipper dredge. As a matter of investment, the dipper dredge at an initial cost of $40,000 will, under average conditions, do the work of a ladder dredge costing $100,000. The ladder dredge has the advantage of being able to work in deeper water than the dipper dredge, and for work of great magnitude, is more economical than the grab-bucket dredge. The ladder dredge is sometimes provided with

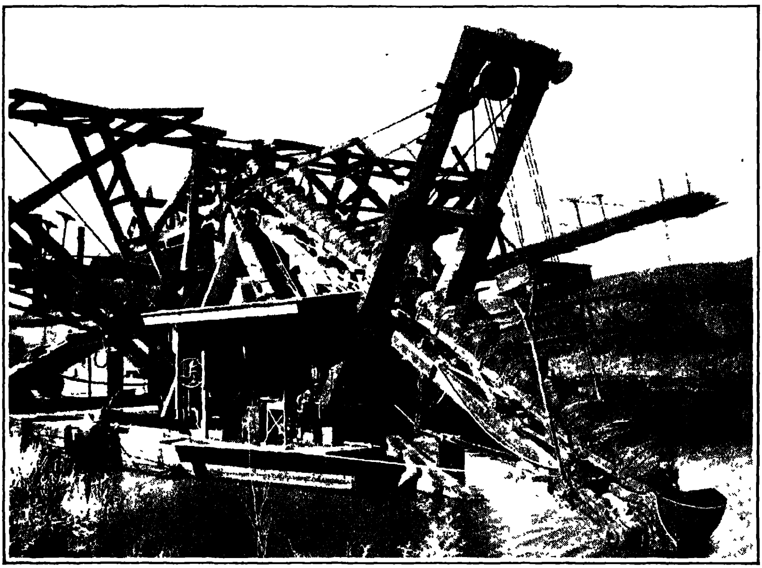


(*Courtesy of Norbom Engineering Co.*)
FIG. 34.—Plan view of hydraulic dredge.

several cutters for the breaking up of rock, previous to its removal by the buckets. This arrangement is very efficient in the excavation of material too hard for the bucket chain to handle without loosening.

Ladder or elevator dredges may be classified as to the location of the elevator—namely, bow-well and stern-well—and also as to the method of disposal of the excavated material—hopper loading and barge loading.

The ladder dredge consists essentially of a hull on which are placed the operating machinery and the excavating machinery. The former comprises the engines for the operation of the bucket



(*Courtesy of U. S. Reclamation Service.*)

Fig. 35.—Ladder dredge.

chain, the belt conveyors, the hydraulic monitor, the spuds, etc. The latter includes the ladder frame and ladder or bucket chain and the method of disposal of the excavated material, consisting of a hopper and a discharge channel, or of belt conveyors. From an inspection of Fig. 35 it will be seen that the principal feature of the excavating equipment is the ladder, which is a framework over which moves the bucket chain. The buckets have a capacity of from 3 to 15 cu. yd. each, and are placed at intervals of from 3 to 6 ft. along the chain. The ladder can be

raised or lowered so that the buckets scrape the bottom and front of the excavation, removing and bringing up material, which is deposited at the top of the ladder, on to belt conveyors or into hoppers.

Some ladder dredges are provided with a hydraulic monitor, which is useful in the washing down of high banks.

Field of Use.—The ladder dredge is efficient in the excavation of all classes of material from silt to the softer stratified rocks, in deep water, and over large areas where there is plenty of space for the maneuvering of the dredge from side to side. It cannot work to advantage in long narrow channels or over restricted areas.

As the ladder dredge is generally a specially constructed machine to meet certain requirements of removal and disposition of material, it is impossible to give a definite statement as to output. In clearing a channel in Boston Harbor, Mass., a ladder dredge operated by steam engines of about 700 h.p., and equipped with buckets of 1¼-yd. capacity, made an average daily excavation of 8000 cu. yd. of clay, gravel, and hardpan at a maximum high tide depth of 50 ft.

Since 1911, a self-contained, self-propelling, twin-screw hopper dredge has been removing stiff boulder clay, soft rock, and conglomerates at the rate of about 1300 cu. yd. of soft material and 500 cu. yd. of hard material per hr., at a 50-ft. depth. The excavating equipment consists of a steel ladder with buckets of 54-cu. ft. capacity for soft material and 34-cu. ft. capacity for hard material. The dredging speed is 18 buckets per min. in soft material and 14 per min. in hard material.

7. Well-point Method of Excavation.—The well-point pumping system has been successfully used in the removal of water from sand, where the use of sheet piling would ordinarily be slow and costly. The system consists in the building of a line of pipe of from 3 to 6 in. in diameter, about the excavation, and tapping into this line, at regular intervals, vertical pipes with well points at their lower ends. These vertical pipes are jettied into place and to the desired depth. A pumping plant is connected to the pipe line and removes the water from the material within the area to be excavated.

A notable example of the use of the well-point system in the excavation of wet flowing soil was in the construction of the 16-story Ambassador Hotel Annex at Atlantic City, N. J.[1] The excavation was 150 ft. by 300 ft., with a general depth of 18 ft., and a depth of 42 ft. in machinery and elevator pits. Surface water was encountered at a depth of 3 ft. and tide water at 10 ft. below street level.

[1] From *Eng. News-Record*, Nov. 4, 1920.

The pumping system consisted of the following: 1, 4-in. main pipe line extending around the area at a depth of 2 ft. below street level and about 6 ft. outside area boundaries; a series of well-points 6 ft. long, attached to 20-ft. lengths of 2-in. pipe, which were connected every 4 ft. to the 4-in. main by 2-in. flexible metal steam hose, and four triplex pumps located at the corners of the excavation. Duplicate pumps were provided as a reserve, and valves were placed so that sections of the system could be shut off, and greater intensity of power concentrated at points needed. The well points were jetted into place by a 3-in. water line. Pumping was continued for three months after the concrete foundation walls were completed.

SECTION 3

FOUNDATIONS

FOUNDATIONS IN GENERAL

By Lazarus White

1. Preliminary Investigations.—Preliminary to the design of the foundations, adequate investigations should be made as to the character of materials. Also the depth to rock should be determined if the rock is within reach. In the writer's experience, failure to make proper investigations has proved very costly to many investors. If a structure is to be placed on spread footings, adequate tests should be made of the bearing value of

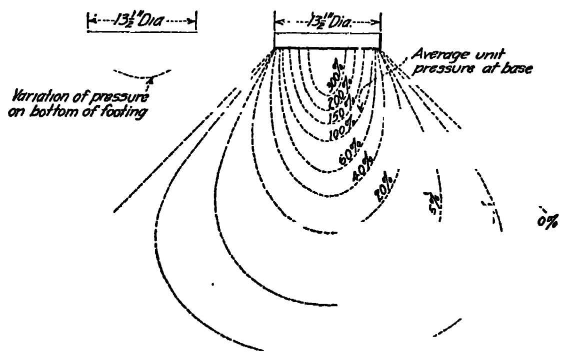


Fig. 1.—Distribution of pressures through soil beneath a footing (13½ in. dia.) as determined experimentally—known as "Bulb of Pressure."

the soil. If a pile foundation is indicated, and data for similar situations is not obtainable, loading tests should be made on the first piles driven, preferably on a group of piles. Pile formulas should not be trusted as they are very misleading.

2. Distribution of Pressure below Foundations.—To understand properly how structures are supported by earthy ground, Fig. 1 should be studied. It should be noted that the pressure is far from uniform and varies in the case given from three times the average pressure at the center of the foundation to zero at the

edge. Tests show that the greater the area, the greater the variation within the area. All of the pressure lines fall inside of a 45-deg. slope (an old assumption), but *nearly* all are contained within a 60-deg. slope to the horizontal. A correct understanding of a spread footing and the transmission of loads beneath will enable one also to understand all other types of foundations, such as caissons and piles.

Attention is called to the fact that the diagram of intensities of pressure here given is true only for the diameter of footings given and that footings of larger area will have different diagrams. With areas varying from 0.4 sq. ft. to 8 sq. ft., intensity of pressure under center of loadings for the same average unit pressure varies almost as the square of the diameter of the footing, in accordance with Professor Enger's formula. Pressures vary according to a parabolic curve from center to edge, but for large areas the curve probably consists of two parabolas tangent to a horizontal line at the center.

Caissons and piles merely transmit the load to lower levels where a firmer stratum is reached or spread the load over a greater area than is practicable with the simplest types of spread footings.

Every foundation can be resolved into units, each of which transfers its load through the material below in a manner similar to that of Fig. 1. Within the limits of the figure, the soil is compressed. The overlapping of the pressure lines causes the variation in pressures noted. For a pile or a caisson foundation the maximum result is obtained when there are enough units to fully employ the entire area available at the site. The ultimate value of the foundation is the entire area times the bearing value of the soil at the depth where the load of the structure is to be ultimately carried. Driving more piles, for instance, within the area beyond this point will do little good, nor will it avail one to count upon any frictional value of these additional piles. The friction on the sides of the piles is only of value insofar as it aids to transmit the load to a lower level.

3. Bearing Power of Soils.—Except for hard rock, cemented gravel, or hard dry clay, knowing the name and class of material in advance of tests does not help much, and assigning bearing values merely by name is very dangerous.

The bearing values used should have some relation to the allowable settlement which will not damage the superstructure,

and tests should always contain enough information to form an intelligent opinion. Such a test made under direction of the New York Bureau of Buildings is shown in Fig. 2.

4. Spread Footings.—Where material near the surface of the ground in excavations made for cellars or basements is such as to

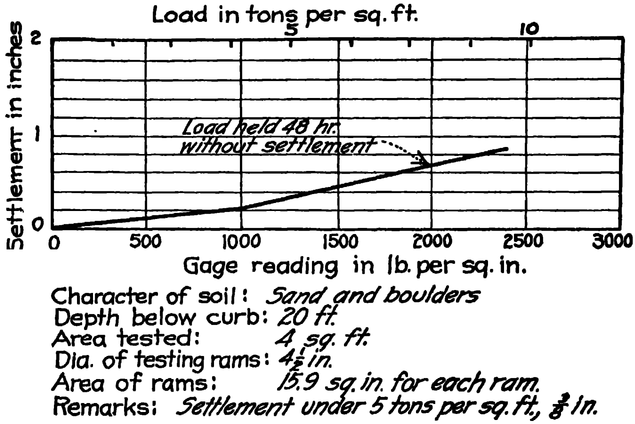


FIG. 2.—Record of soil test made under direction of Bureau of Buildings of Manhattan, New York City.

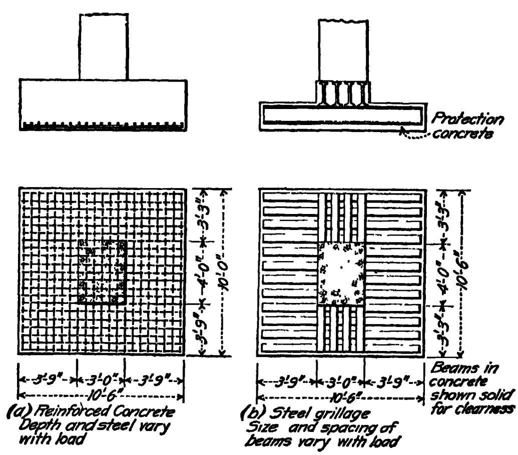


FIG. 3.—Typical spread footings.

allow a value of several tons per square foot, then a spread footing is usually the most economical, but not necessarily so, as in city streets where extensions of foundations more than one

foot beyond the building line are not allowed, expensive cantilever footings may run the cost above that of other types.

Two types of single or isolated column footings are shown in Fig. 3. These footings are usually designed on the assumption of uniform pressure per square foot obtained by dividing total load by total area. This assumption is much on the safe side as the pressures diminish from center outwards, as indicated by Fig. 1.

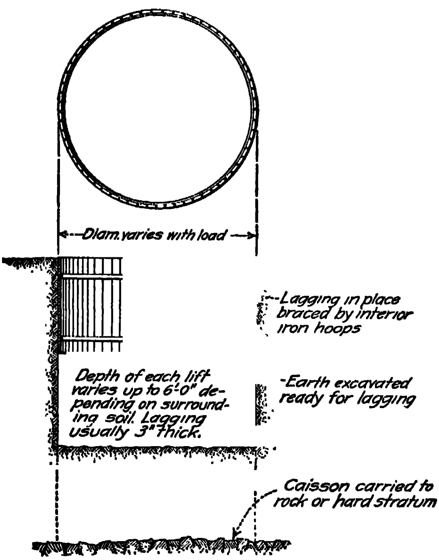


FIG. 4.—Chicago open caisson.

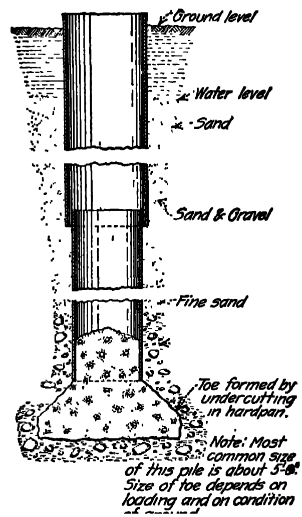


FIG. 5.—Gow pile.

Although a footing designed according to the assumption of uniform pressure may be over designed, settlements greater than expected may occur.

5. Open Piers and Caissons.—The simplest method of making available better material found at a depth below the preliminary level of excavation for the structure is by digging open pits or caissons. There are three methods in vogue. One known as the Chicago open caisson, shown on Fig. 4, is extensively used in Chicago to reach rock or hard stratum below, penetrating mostly clay of various degrees of hardness.

In New York the horizontal or vertically sheeted pits are extensively used to penetrate sandy material in order to reach rock or "hard pan."

In Boston, the "Gow" pile (see Fig. 5), really a caisson, is used extensively to penetrate material—mostly clay—overlying hard clay or cemented gravel.

Each of the above methods are suitable for their (or similar) localities, and, next to spread footings, are most economical for foundation purposes.

6. Pneumatic Caissons.—Where it is not practicable to dig through wet ground in the open in order to reach rock or better material below the main excavation, pneumatic caissons are

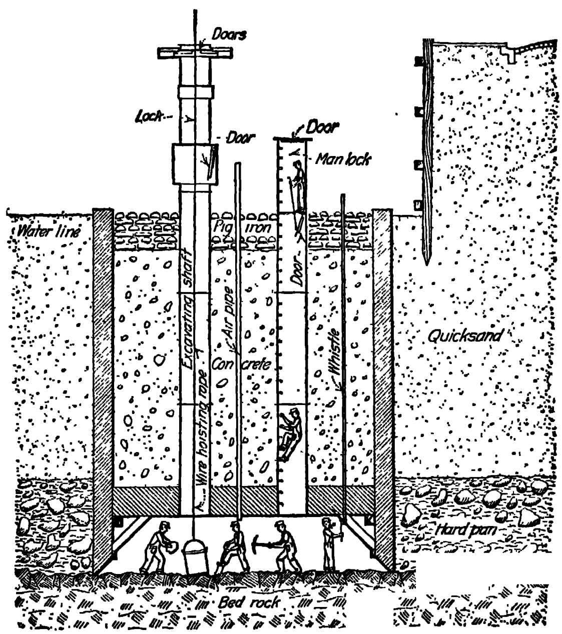


FIG. 6.—Pneumatic caisson sunk to bed rock.

employed. These are simply masonry boxes sunk by means of compressed air to the desired depth and then filled with concrete. The carrying capacity of each caisson is its area times the bearing value of the rock or other material upon which it is founded. They have been sunk to depths where it was necessary to employ about 50 lb. of air to balance the exterior water pressure in the soil corresponding to a depth of about 100 ft. A typical caisson is shown in Fig. 6.

Pneumatic caissons are, of course, admirable for very heavy foundations in difficult situations, such as for bridge piers and for

monumental and expensive structures. They are particularly applicable where it is desired to use them to form a cofferdam within which a deep open excavation to rock or hardpan may be made, enabling the placing of the interior footings in rock or hardpan.

Great improvements of late years have been made in the methods of sinking open piers and in various pile and cylinder foundations, so that pneumatic caissons are rapidly being supplanted for all but the most monumental structures.

7. Wooden Pile Foundations.—Formerly, where soft ground was encountered, wooden piles were most frequently used and this still is very common, but they are being rapidly supplanted

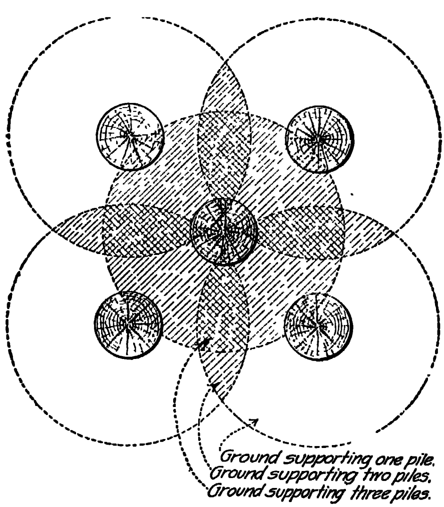


FIG. 7.—Group of piles closely spaced, showing overlapping of supporting areas. Note that center pile brings no additional capacity to pier.

by concrete and steel piles or cylinders. Where wooden piles are continually below water level, they last indefinitely, but rapidly decay when exposed to the air. Much damage has been caused by the drawing down of the ground water level and the exposing of wooden piles. The water level of cities is nearly everywhere being lowered so that wooden pile foundations should only be used close to large bodies of water which cannot be drained down. Wooden piles when driven to a good bottom are good for 10 to 15 tons each when driven on about 3-ft. centers. As the cheapest

trees are used for this purpose, they should be driven very carefully, overdriving especially being avoided. This danger is greater than ever as larger and more powerful hammers are used. There is no known formula which will give a true value for a driven pile so that the true value can only be obtained from tests or previous experience. Tests on a single pile of a group may give a deceptively large value as the pile tested may throw part of its load on its neighbors. After a certain number of piles are driven within a given area, it does little good to drive more as the additional piles merely bear on the same ground as their neighbors, as shown in Fig. 7.

Wooden piles are driven in groups and capped with concrete, plain or reinforced, or driven in uniform spacing over the entire area and capped with a layer of concrete. To insure the permanence of the piles, they are cut off at low tide or the water is pumped down below its permanent level for this purpose. This necessary precaution often adds very materially to the cost of this type of foundation and may make another style of pile desirable or necessary.

8. Concrete and Concrete Steel Piles.—Where ground water conditions are such as to make wooden piles undesirable and where higher values per pile are sought, there is quite a choice of concrete and concrete steel piles or cylinders. It is in this direction that the greatest progress has been made in recent years.

8*a*. Concrete Piles.—Reference should be made to the chapter on "Concrete Piles."

8*b*. Steel Tubes.—Steel tube or pipe foundations have been very extensively used for the past 25 yr. in the vicinity of New York and to a considerable extent elsewhere. They consist of standard steel tubes or pipes from 10 to 16 in. in diameter and usually 3/8 in. in thickness. Factory lengths average about 20 ft. Where more than one length is necessary, they are connected by steel inside sleeves.

They are usually driven open end by steam or air hammers to rock or hardpan and excavated by means of compressed air blow pipes or water jet, and concreted. When well seated, they give the highest load values of any form of pile foundation. Experience shows that very little of the steel shell rusts, some having been uncovered after a long period of years. Their load values may be computed as follows:

For piles consisting of steel tubes filled with concrete, the tubes shall have a diameter of 9 in. or more and a thickness of not less than 5/16 in. The ends of each tube shall be faced perpendicular to its axis. Splices shall be of an approved design and not more than one splice shall be used in the total length of the pile. The length of any such pile shall not exceed forty times the inside diameter of the tube. Such piles shall be driven to a full bearing on rock. The allowable load on any such pile shall not exceed 500 lb. per sq. in. on the concrete and 7500 lb. per sq. in. on the steel, provided that in computing the effective area of the steel the outer 1/16 in. of thickness shall be deducted from the thickness of the tube. No interior steel reinforcement shall be used.[1]

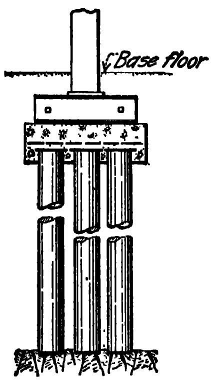


FIG. 8.—Typical steel pile foundation—known also by trade name "Hercules" and "Tuba Steel." Piles are usually 15 in. dia., 3/8 in. thick, concrete filled.

Steel tube foundations are generally used where rock is within 40 ft. of the bottom of the footing, but have been used for depths of 70 ft. Due to the high value obtained for each tube (about 100 tons for shell with 15-in. diameter and 3/8-in. thick), the column footings are very simple and expensive cantilever footings are avoided. Owing to the great strength of the tubes, they can be driven through ground where it would be very difficult to drive other types of piles. A typical design is shown in Fig. 8.

9. Pretest Foundation.—A new type of foundation known as the Pretest System (patented) has been in use for the past 4 yr. for some important buildings. This type of foundation consists of hydraulically installed steel tubes (14 to 20 in. in diameter) concrete filled.

This type of foundation is unique in respect that it enables the simultaneous construction of the foundation and superstructure with a consequent saving in time.

It has been used where rock was at such a great depth as to preclude the use of ordinary piling or caissons, and where the overlying material, a saturated mixture of fine sand and clay, or the proximity of neighboring structures or streets, would not allow the use of an economical spread footing.

The cylinders are usually installed by hand, the footing cast a

[1] From New York City Building Code.

few feet above the top and while the construction of the building is progressing, cylinders are successively tested by hydraulic jacks to a 50 per cent overload and wedged to the footing without allowing any release of pressure. In this operation every cylinder is tested, thus providing a known factor of safety to the entire

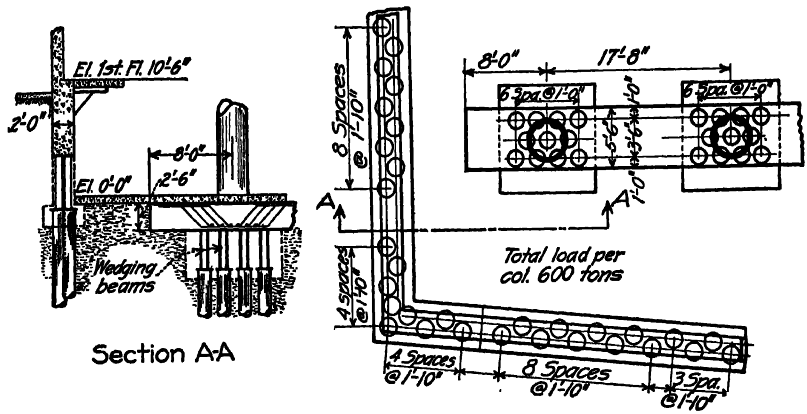


FIG. 9.—Typical pretest foundation (patented) as used for 17-story reinforced concrete building. Diameter of cylinders—14 to 20 in. Loading—350 to 500 lb. per sq. in. of cylinder cross-section.

building. The foundation is completed when about 90 per cent of the weight of the building is in position. Cylinders are usually figured from 30 to 50 tons for diameters of 14 to 19 in. This type of foundation is ordinarily more economical than the steel tubes to rock when rock is more than 30 ft. below the footings.

A typical installation of Pretest piles is shown on Fig. 9.

FOUNDATIONS IN WELLS OR PITS

By James C. Meem

10. Soil Characteristics.—In discussing foundations in wells or pits consideration must be given, first of all, to the different characteristics of soil, since the method or type of foundation construction to be selected is so essentially dependent on these characteristics. Disregarding for the time the ordinary soil classification, soils may be divided into five classes according to their adaptability to methods of constructing foundations in wells or pits:

(1) Non-cohesive—as hot, dry sand; grain may also be included under this head.

(2) Semi-cohesive—normally dry (moist) soil.
(3) Cohesive—as hard clay or solid rock; soil which does not break or tend to break down under ordinary operations. When the operations are sufficiently large, as n mining, to cause this breaking, the soil should be classed as semi-cohesive—that is, solid rock or hard clays under these conditions become granular aggregates. In ordinary pit or well sinking, however, these soils retain their cohesive or solid characteristics.
(4) Semi-aqueous—this class includes all granular soils in which the water or liquid is retained in the voids by confinement or by pressure from the outside, and from which it may be excluded by releasing the confining media, such as by drainage, or by pumping, or by internally applied pressure as compressed air. When water is so incorporated in the soil that it is virtually a chemical compound, or when it does not readily drain out, as in plastic clay, it should be included under Class (5).
(5) Aqueous—this class includes pure water or the equivalent, plastic clay, etc., in which the specific gravity of the mass is virtually its weight. All soils in this class—as is the case with water—exert, when confined, pressure in all directions, and may be displaced by ordinary pressure when not confined.

11. Methods of Well Sinking.—The simplest type or method of well sinking is the old well-digger's method. This consisted of excavating a square pit (about 6 × 6 ft.) as far as possible ahead of the protection, which was made of small tree trunks split or solid—those in the north and south ends alternating with those in the east and west walls, first as braces and then as rangers. When water was reached, baling was resorted to and the method continued as long as was consistent with safety. After this, if it was necessary to go deeper, a square set of sheeting was set up and by driving this sheeting ahead of or with the excavation, the excavation was usually completed with safety, and was then walled up with dry rubble.

Contractors, more particularly those operating in lower New York, have adapted these methods of pit sinking from those of the well diggers, and, in connection with subway work in New York City, many thousand pits have been sunk in order to extend or deepen the foundations of buildings to be underpinned.

Briefly, the operations consist in excavating below the last set or ring for placing a new set. Ordinary 2 × 8-in. rough board sheeting is used and the pits average 5 ft. square, the board lengths being alternately 5 ft. and 5 ft. 4 in. The north and south sides alternately brace or are braced by the east and west sides as noted. The secret of both success and economy of

operation by this method is in keeping the sheeting tight and well backed, and seeing that no voids or loose ground are left behind. For this reason the work should be entrusted to skilled workmen only, or men who have been properly instructed. When this is assured, the method is safe, rapid, and economical.

One of the most common methods of protecting pits during excavation consists in driving 2-in. sheeting in short lengths of from 5 to 7 ft., through square sets, consisting of alternating rangers and braces backed by wales. The sheeting is driven with outward inclination sufficient to clear the next set. This method creates and leaves a very unsatisfactory condition at the corners, due to this outward inclination of the sides, which is usually cared for by makeshifts, as salt hay, or short light planking set in as a horizontal backing, or both. When the pit is sufficiently shallow to admit of driving sheeting or interlocking sheet piling for the full length in one set, very satisfactory results may be obtained by this method, even where water is present in the soil to a considerable extent. In the latter case, in water-bearing material, interlocking steel sheet piling gives better results than wood owing to the tendency of the latter to separate at the corners. The pit may be of square or preferably of round cross-section. In this connection it may be well to note the difference between the terms "sheeting" and "sheet piling" as used here. Sheet piling defines the tongue and groove wood or interlocking steel types driven entirely in advance of the excavation—while by sheeting is meant plain board or steel protection set in or driven as the excavation proceeds.

Where self-contained cylinders or squares can be used, and sunk in one length, or as built up, it is usually cheaper and more satisfactory to use them. They may be of steel or concrete, or a combination of the two.

Very satisfactory results have been obtained, especially in lower New York City, in driving steel cylinders in one or more short lengths in the bottom of an excavation or pit, below which ground water is present. These sections are ordinarily in lengths of from 2 to 6 ft. and made up of steel from No. 12 gage to ½ in. in thickness, according to the requirements, and from 9 to 30 in. or more in diameter. They are connected by an overlapping inside ring of steel or by special inside collars of steel or cast iron. They are driven by a winch-actuated hammer, or by a hydraulic pump (if reaction may be had) actuating independ-

ent rams; and are cleaned out, as the work progresses, by augers, scoops, or more rapidly by dwarf orange-peel buckets.

Telescopic cylinders have been used more or less successfully, the principal advantage apparently being found in overcoming additional skin friction. As this additional skin friction is relatively small, this advantage is minimized by the additional cost of the larger top section of the telescope. Where the cross-section is larger than 30 in. and the depth too great for driving steel piling in one length, an open reinforced concrete cylinder may prove best adapted to the requirements, providing the ground is soft enough to allow the cylinder to be sunk as built up under its own or small additional weight, when cleaned out.

12. Wells or Caissons Sunk with Compressed Air.—Where the pit can not be sunk without the use of compressed air, the open reinforced type of cylinder or square is best and most economically adapted to this condition. The bulkhead is usually placed as near the bottom as practical to allow sufficient head room in the working chamber and the load required to sink is placed above the bulkhead and around the vertical lock of which small caissons have usually only one for men and material. The walls may be completed before sinking or be built up while sinking is in progress. In caissons small enough to be in the well class, skin friction (as noted later) is not considerable, and if no binding or irregularities occur there should be little difficulty in sinking. A small additional weight above that required to overcome the difference between normal weight and air pressure is all that should be needed. Of course where the process does not jeopardize the work itself or adjacent structures, blowing (i.e. dropping the air pressure to allow the caisson to sink more rapidly) may be resorted to. This unbalancing of the pressure is, however, always attended with the possibility of and danger from an inflow of soil from the outside, as well as consequent increased difficulties of sinking and the incidental possible injury to the air workers.

Where owing to lack of head room or for other reasons it is sometimes not possible to sink a caisson by this method even though compressed air is required, a fixed lock may be used. The preferred method in this case is to sink a small cofferdam or pit in the open and somewhat larger than required for the air work to the water-bearing soil. Above this a roof or bulkhead is placed with a vertical or horizontal lock or locks in place. This bulkhead is then backfilled or weighted to more than

balance the requisite pressure. From this working chamber under compressed air, the smaller caissons may be sunk as in the open as described.

13. Relation of Method to Soil.—The proper adaptation of the methods described to the different classes of soil is of great importance. Taking the soils as described in their order:

(1) *Non-Cohesive Soils, as Hot Dry Sand or Grain.*—It is obvious that any well-digger's method or adaptation—such as excavating sufficiently deep for the setting in of horizontal sheeting—is not applicable, nor is that of driving sheeting through short sets practicable, owing to the difficulty of preventing the soil from running through the corners. The method to be selected therefore is one in which the protection is driven in advance of the excavation and is always tight at the vertical joint or faces, such as self contained cylinders, squares, or rings of sheet piling driven in one length. Where no openings along vertical faces or joints are left through which the dry material can run, no danger need be apprehended from excavating to the bottom of the well or caisson, as there is no tendency to displacement by pressure, within the limits of application ordinarily possible.

(2) *Semi-cohesive Soils.*—This class covers a large percentage of the soils in which structures are built. In almost all cases, soils of this class stand up fairly well. Excavation may be made ahead of the protection, and the well-digger's method, or some adaptation of it, may be used to greatest advantage. It may sometimes happen that soil of this class is so granular and nearly non-cohesive that horizontal sheeting planks of ordinary width may not be used and, in this case, planking or reinforced steel sheets of 6- or even 4-in. in width may be substituted for the 2×8 in. or 2×10 in. ordinarily used. Methods calling for interlocking sheeting in short lengths, circular or square wells of interlocking sheet piling in one length, telescoping cylinders, or cylinders as previously described, are all applicable to this class of soil.

(3) *Cohesive Soils Including Medium Hard or Hard Clay and Rock.*—In hard clay, pit excavation may be made without any protection becoming necessary if the pit does not have to stand too long. The length of time a pit excavated in hard or medium hard clays may stand is very indefinite and may range from a few minutes to months, or even years.

Pits have been noted which have stood for years with very little spalling, whereas in many of the clay soils around the Great

Lakes the protection should follow the excavation within a few hours at the most. This is due in some cases more to seepage than to spalling, although before excavation there may not be enough moisture in the soil to cause erosion.

In cases where the rock or clay is so nearly non-cohesive as to spall or tend to break down with the progress of the excavation, the soil should properly be classed as semi-cohesive.

(4) and (5) *Semi-aqueous and Aqueous Soils.*—Wells in semi-aqueous soils may sometimes be sunk by driving some form of close sheeting or sheet piling ahead of the excavation, and pumping or baling as long as the inflow of water is not great enough to cause erosion or the washing out of the soil under and back of the protection. When this condition exists in semi-aqueous soils, or where there is sufficient wet clay or other lubricant in the soil to cause it to be displaced under ordinary pressure (as heretofore noted under aqueous soils), compressed air must ordinarily be resorted to, although it is possible in some instances in plastic soils to sink wells in the open by working rapidly and putting in the protection before the soil has time to flow in due to outside pressure. It is preferable and much safer, however, to operate in such soils under a fixed lock, with compressed air, by methods similar to those used in hard or medium clay.

Attention is called to the difference between the action of compressed air in sinking wells through granular water-bearing soils and through dense plastic soils. In granular water-bearing soils the water is simply driven out of the soil, thereby preventing erosion, and since the soil has little cohesion it must be sheeted and braced and generally treated as the same soil when dry and under normal air would be treated. In dense plastic soils the compressed air acts as a brace—that is, in such dense soils the air can not get into the voids behind the sides of the excavation to balance the outward air pressure. It is thus seen that plastic or treacherous soils may frequently be handled more safely and economically under compressed air without bracing, than firmer granular soils which not only must be sheeted and braced, but even under compressed air must have the protection tight and driven ahead of or coincidently with the excavation.

14. Resistance Encountered in Sinking Wells and Safe Foundation Loads.—Attention is called to the generally well-known fact that the bearing value of soil is very much greater at the base of a cylinder or confined pit than the same area of

soil unconfined at or near the surface. That this increased bearing value is not due appreciably to skin friction has been demonstrated in numberless tests, showing that smooth bore steel cylinders driven practically to refusal with the soil in place, sink, when cleaned out to the bottom, under a comparatively small load until a new soil plug is formed.

Fourteen-inch round, open cylinders (approximately 1 sq. ft. in area) have frequently resisted a measured pressure of 60 tons without sinking, in soft ground, which when cleaned out to bottom sank readily under a load of less than a ton. This is positive evidence that skin friction is a non-appreciable factor in such high resistance factors and should not be relied upon to any considerable extent, particularly in small cylinders.

The sinking of a caisson under extreme extra weight, or unexpected difficulty in pulling a pile is often confused with high skin-frictional resistance, because of binding, or large protuberances, etc. As friction is always an element and percentage of the actual pressure it must be always less than that pressure. As the pressure per square foot on small caissons, pits, or piles cannot be very great, the skin friction per area must be always less. Taper, knots, etc., are frequently assumed as increasing skin friction when in reality they are adding to the bearing value. Broadly stated, all horizontal, or horizontally projected areas should constitute bearing area and all vertical and vertically projected areas should constitute skin frictional area.

As the large increase in the bearing value of soil confined at the base of a pit, caisson, or pile, cannot be attributed to friction, it must be accounted for by lateral transmission and dissipation of pressure through soil. It has been demonstrated by tests[1] that this pressure transmission takes the form of a pear-shaped bulb, called the *bulb of pressure*. In this connection it is well to note the decided advantage which would accrue to engineers if the so-called bearing value of soil was given in terms of the expected settlement per square foot under a given loading. In order to place a load of 4 tons, for example, on a square foot of soil without any expectation of settlement, the area should first be loaded to 6, 8 or 10 tons, or at least compacted by the dropping of a heavy hammer, after which the 4-ton load may be safely placed. Under these conditions solid concrete pits 4 or 5 ft. square could be designed to carry 6 or 8 or even up to 12 or 15 tons per

[1] See Art. 2, p. 89.

sq. ft. safely, while 14-in. cylinders having an area of 1 sq. ft. (and up), tested first to a load of 40 or 50 tons, should carry 25 tons permanently thereafter without serious settlement.

15. Placing Concrete.—The preferred method of placing foundations in a well pit or small cylinder is by "shooting" the cement through a chute, if in the dry, or through a tremie, or a bottom-dumping bucket if the structure is not unwatered.

It will probably be found more economical to concrete even a 5 × 5-ft. pit solid than to attempt to use forms to reduce the bulk of concrete, though this is a matter to be determined by the engineer on the ground. Reinforcement is rarely called for under any of the conditions noted, but this too is a simple matter of judgment for each case.

Where small cylinders are sunk to rock without being unwatered, no apprehension need be felt because of the fact that the rock may not be level nor thoroughly cleaned, providing ordinary care is used in getting out the softer soil and in placing the bottom concrete.

COFFERDAMS[1]

By James C. Meem

16. Definition.—Not with any view of creating a new definition but to avoid misunderstanding, a cofferdam may be defined as an artificially protected area from which water and (or) soil is removed for the purpose of placing a structure therein.

17. Types of Cofferdams.—Primarily cofferdams are of three general types:

(1) Self-contained structures—usually circular structures of plain or reinforced concrete, or steel piling. Small cofferdams of this type have already been described in the chapter on "Foundations in Wells or Pits."

(2) Cofferdams with braced or supported walls.

(3) Cofferdams with self-sustaining walls, other than those wholly self-contained as noted in (1)—also known as cellular cofferdams.

18. Soil Pressure on Cofferdams.—Since cofferdams, particularly Types (2) and (3), are usually of large area and depth, the pressure of the soil on their walls constitutes a very considerable factor in their design and a short discussion will therefore be

[1] See also chapter on "Sheet Piles," p. 198.

given, relating to the pressure of dry and water-bearing soils against cofferdams or other walls.

If in Fig. 10 we assume that *AO* is the braced wall of a trench or cofferdam, and that, if it be removed, the normally dry sand behind it will slide away to the line or plane *OC*, then *OC* is the line or plane of repose of this soil. If then the face *AO* be restored

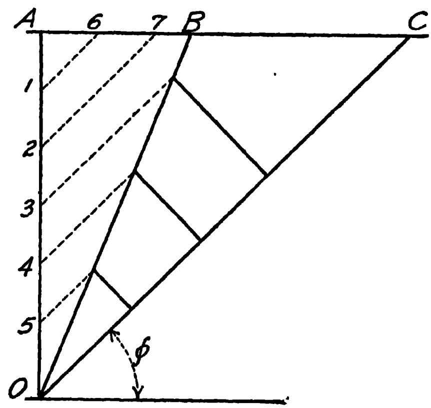


Fig. 10.

and the area *AOC* again be backfilled, it is undoubtedly true that no soil below *OC* can exert any pressure on *AO*, but that all of the soil in the area *AOC*, being restrained from sliding by the wall, does exert some pressure on it. If the boards *A*-1, 1-2, 2-3, etc., are removed, one at a time, the areas or volumes above the slope line at these points tend to slide away, and therefore before removal must exert pressure proportional to their areas. With the boards or wall in place, the tendency of the soil is, normally, to move toward the operating face where some small movement or leakage is probably occurring. When a sufficient depth has been reached, as *A*-4, Fig. 11, the soil tends to establish itself by forming a natural arch between *A*-4 and *CG*. If, for instance, the line or surface 4-*D*-*G* (Fig. 11) should be lagged and attached to the mass by bolts bearing through large washers on the surface, all the soil below 4-*D*-*G* could undoubtedly, and with perfect safety, be removed, the soil arching between *A*-4 and *CG*.

The soil pressure per linear foot of face on *A*-1 (Fig. 10) may be determined by multiplying the volume represented by the

area A-1-6 by the weight of the soil per cubic foot and dividing by the tangent of the angle of repose (ϕ). In a like manner the soil pressure on 1-2 may be determined by considering the area 1-6-7-2. The point of application of the pressure coming from

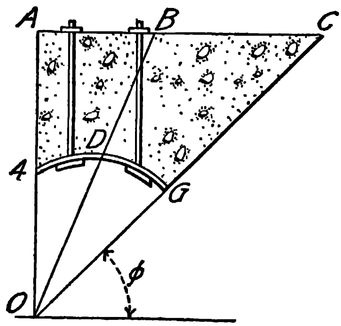


Fig. 11.

each of these areas is where a line parallel to the plane of repose passing through the center of gravity, intersects the vertical face or wall.

The assumption is arbitrarily made that the line BO bisects the angle between the angle of repose and the vertical and therefore

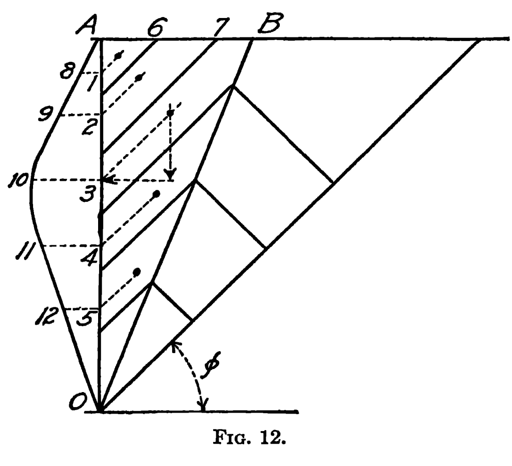


Fig. 12.

that all the soil in the area above presses against the wall and all below rests on the plane of repose. The actual determination of just where and what this line is has not been made, nor can it be except by experiments on a large scale, but it probably is not far

from correct to assume it as noted, and in any case it is believed the error, if any, will be on the side of safety.

By determining the pressure for each area referred to above (*A*-1-6, 1-6-7-2, etc. Fig. 10) and platting each result to the left of its point of application, a graphical curve of pressure is developed, as shown in Fig. 12, with its maximum thrust at a point slightly above the middle of the vertical wall.

Coming now to the question of the pressure of water and of aqueous or water-bearing soils, the pressure curve for water can

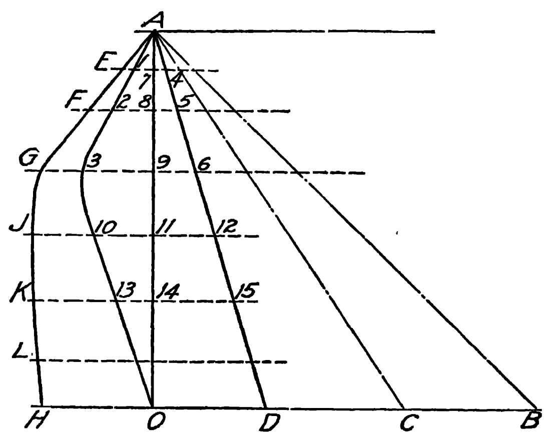


Fig. 13.

first be plotted, as in Fig. 13, where *AO* is the wall of a cofferdam restraining water and *AB* is the graphical line of pressure, the pressure at *O* being represented by the horizontal ordinate *OB*.

Since water weighs only 62½ lb. while the weight of earth is approximately 90 lb. per cu. ft., the point *C* is located, in which *OC* is approximately 70 per cent of *OB*, *AC* being the relative pressure curve. If then from Fig. 12, the curve *A*-1-2-3-10-13-0 is reproduced, the relative pressures of normally dry soil and of water are shown relatively and to the same scale.

In the case of semi-aqueous or water-bearing soils—that is, where the voids of the soil are wholly filled with water which may be expelled by pumping, drainage, or air pressure—the pressures of the soil and water while acting in unison are wholly distinct. It is not correct to assume that the full pressure of the water is exerted plus the full pressure of the soil measured by its weight in water—nor is it correct to assume that the pressure is

due to that of an aqueous mass of a specific gravity equal to the weight of a volume of the sand and water together.

It would appear, then, that as long as there is no movement of the water sufficient to cause hydraulic action or erosion of the soil, that the presence of the water does not change its pressure effort, except to a relatively small degree in its cohesive element and also in reducing its pressure-weight element by the amount of its added buoyancy. If, however, it is realized that soil and water can not produce pressure over the same area at the same time, it can be assumed that some areas of soil must be in contact with the wall and lead back through somewhat tortuous areas (which may be considered equivalent to solid columns) to beyond the pressure areas, and that between these areas are leads or areas of water giving full pressure equivalent to the hydrostatic head of each over this reduced area. Under this assumption the loss of weight of the soil due to the water is disregarded and the combined pressure is that of the soil of full weight as if normally dry, plus the water pressure acting through the voids of the soil. If it is assumed that these voids for safety are 50 per cent, then the intensity of pressure at any depth is represented by the horizontal distance between *AO* and *AGH* in Fig. 13. *OD* is made 50 per cent of *OC* and *AD* is the curve or line of pressure due to the water in the soil. *A*-1-2-3-10-13-0 as noted measures the soil pressure and by plotting *E*-1 equal to 7-4, *F*-2 equal to 8-5, etc., the curve of combined pressure *A*-*E*-*F*-*G*-*J*-*H* is obtained. When the water is above or below the surface of the soil, the pressure curves may be separately plotted and combined as above.

The fallacy of attempting to measure the pressure of soils or the combined pressure of water and soil on cofferdam walls by means of gages through the walls should be apparent to anyone who realizes that in normally dry soils, areas as large as 1 ft. square may be frequently left or cut in the sheeting without any danger or any pressure being noticed, even though the braces and rangers over the whole area are under heavy stress. It is also true that the bottom plank of a horizontally sheeted trench in normally dry soil may be placed or removed without showing pressure stress, and this may even be done in soils of firm sand or gravel where there is a seepage of water not sufficient to cause erosion. On the other hand, in aqueous soils such as stiff plastic clays, the gage may show a much larger pressure per area owing

to the blanket-like action of the clay. In firm, sandy, or gravelly soils as just noted, the gage will ordinarily show water pressure due to the full hydrostatic head and very little else. The only possible way to correctly measure these pressures is by apparatus which will give the pressure on the whole area, as of a tunnel roof or the entire wall of a cofferdam. Until this is done on a scale sufficiently large to be conclusive, all earth pressure formulas will be largely theoretical instead of empirical or practical.

19. Self-contained Types of Cofferdams.—Small caissons of this type are treated in the preceding chapter. It is probable that conditions will not often be found where it will prove economical to design self-contained caissons greater than 20 ft. in diameter owing to the heavy walls which are required, particularly for deep structures.

When, however, it is found economical or expedient to use such designs, concrete, plain or reinforced, should probably be used. The design should call for a reinforced cutting edge as in pneumatic operations with a slope of about 2 vertical to 1 horizontal. If the cofferdam is to be sunk in the open, its weight will probably be found sufficient to sink it when built up as it sinks. It may be mucked out with a clam-shell or orange-peel bucket, leaving the water in place to prevent erosion under the cutting edge and consequent loss of surrounding soil.

Where it has been previously determined that the soil at a given depth is of proper character to bear the foundation, cofferdams of this type may be sunk 3 or 4 ft. beyond the required depth and a protective area of concrete, reinforced if necessary, may be deposited under water. If ordinary care is used in the placing of this protective bottom, the caisson may probably be safely unwatered to place the foundation as called for. It is best, however, to take additional precautions, such as examining for leaks and cracks as the water is lowered, and providing loaded platforms resting on and weighting the bottom through light steel legs, which latter may be incorporated into the foundation without removing the weight until the foundation is in place. If the cofferdam is to go to rock, and soundings have shown that the surface of the rock is fairly regular and level, this type may be used, though it may be necessary to send a diver to clean the rock or to see that no treacherous soil remains, as well as to seal any small openings, which are preferably closed by him with concrete in porous bags.

When boulders are met with in sinking the self-contained or other types they are usually dislodged with high pressure jets, though at times the help of a diver may also be required.

20. Cofferdams with Braced or Supported Walls.—When the cofferdam is to go to rock and the surface has been found to be very irregular in contour, or sloping more or less steeply, the self-contained type is not called for. A preferred type would be a ring of steel sheet piling driven in one continuous (or spliced) length if practicable. If this ring, set true and heavily braced inside and out, before driving is begun, can be wholly driven before excavation is commenced, the operation is greatly simplified. Excavation may then follow the pumping or unwatering, and bracing (consisting of rings built up of segments) may be placed at required intervals as the excavation progresses.

Although the ordinary open trench is not, strictly speaking, a cofferdam, it often happens in large operations that the foundations of structures lie below the water line where the water is easily controlled by pumping, and ordinary sheeting and bracing may be used. In such cases rangers backed by light waling pieces are first laid in position in as deep an excavation as it is reasonable to make without protection, and ordinary sheeting planks (preferably 2-in.) are set and driven as the excavation proceeds, being well braced at the start to insure being driven plumb. If the soil is granular and easily unwatered, this plain sheeting may be used to the bottom. If, however, the soil is aqueous or plastic, light tongue and groove wooden sheet piling should be used, being driven in the same way as plain sheeting. Each set of sheeting (usually in 12 to 16-ft. lengths) is stepped in so that the last ranger of the upper set becomes the waling piece of the next lower set. Short vertical pieces are set in or driven under the braces, in ordinary operations. Where it is necessary to insure absolute tightness in the sheeting, the upper rangers must be set with the braces bearing against inside wales blocked out from the rangers, the blocking being split out as the sheeting is set and driven.

The pumping is done from a tight sump preferably of sheet piling and circular in section. The water should flow or seep to this sump over its top where it can be watched and controlled, rather than through or under it, and the sump should be lowered as the work progresses.

This general type of cofferdam with braced walls also covers

the method of driving interlocking steel or tongue and groove wood piling which is preferably driven in one continuous or spliced length and therefore does not require any offsetting.

In cofferdams or trenches of this type the piling is usually pulled as (or after) the structure is placed and backfilled. The pumping or unwatering and the placing of the foundation is carried forward as heretofore described.

21. Cofferdams with Self-sustaining Walls.—This type is preferably used where the area of the foundation is large and where a considerable percentage of the depth is through water alone. In this type interlocking steel piling arranged to conform to a plan of continuous, connected bays, as shown in Fig. 14, is driven around the area through the overlying soil to rock, or as required.

Fig. 14.

The depth of penetration of the piling is not an essential element of this type of cofferdam, but the two important factors

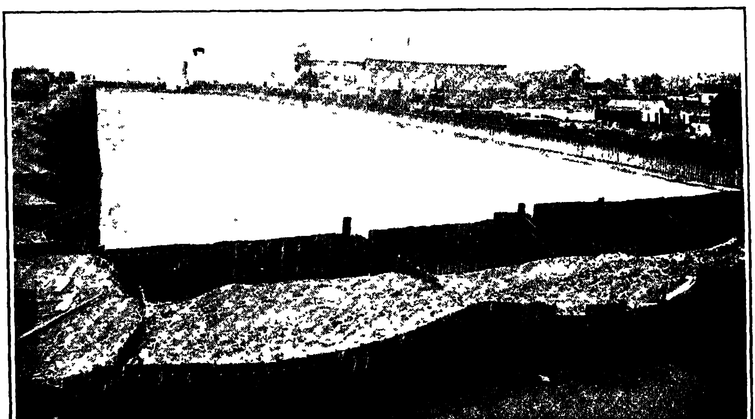


Fig. 14A.—Cofferdam for the U. S. Government ship lock at Black Rock Harbor, Buffalo, N. Y.

are the inward pressure of the soil on the wall, and the pressure of the natural or filled soil in the bays against the inner ring of piling. The outside pressure is measured by the stress diagrams already given for water and soil, and the wall is made sufficiently thick to resist by its own weight or inertia the tendency of these stresses to strain or distort it. The pressure of the soil (and

water contained in the voids of the soil) in the bays is measured in the same way as noted, but the inner ring of sheet piling should be considered in resisting these stresses, as the cables of a horizontal suspension bridge, and a specially designed or tested type of piling should be used in which the strength at the interlock is sufficient to resist the jaw pull. If this inside ring of piling is driven with a belly of 10 per cent of its length between cross walls, its resistance will of course be greater and more readily calculated then if driven "flat."

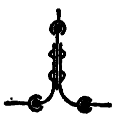


Fig. 15.

A specially designed and tested type of three-way pail should be driven at the end of each cross wall (see Fig. 15). The closures in each case are made by using specially designed closure piles or by designing the closure to be made at the back of each bay (when special care is not so requisite) and by setting the last several piles to close, before driving. Notable examples of this type were the Black Rock cofferdam at Buffalo (Fig. 14*A*) and the cofferdam for raising the Maine in Havana Harbor. In the latter (though not strictly speaking a cofferdam for foundations) the piling was driven in the form of continuous figure-eights in bays to form the walls. The piling did not go to rock but to a depth in the harbor sufficient to prevent the inflow underneath it of soil during the process of unwatering and excavating for the skeleton of the Maine. The figure-eight section of wall is not believed to be as stable as the type shown in Fig. 15, which section was used in the Black Rock cofferdam at Buffalo. The walls there were made up of sheet piling driven in square sets or bays—each 30 × 30 ft. The piling was driven to rock, and the bays, above the normal bottom, were back filled with dredged soil. When unwatering was commenced, the inside sheet piling face of each bay bellied more than 3 ft., and some alarm was at first felt for the stability of the structure. Under these conditions, however, the piling (with this belly), formed the equivalent of a horizontal suspension cable and the strength at the jaw was found by tests to be more than sufficient to resist the stress due to the pressure of the soil contained in the bays. 12-in., 40-lb. Lackawanna piling with a tested strength at the jaw of over 9000 lb. per linear inch was used in the two cofferdams described after a series of comparative tests of five types of sheet piling.

An interesting type of cofferdam which does not properly belong under any of those noted was recently described in *Eng. News-Record,* Feb. 13, 1921, as having been successfully developed at Atlantic City, N. J. in excavating some 5000 cu. yd. of sand within 100 ft. of the ocean for the foundation of a large hotel. In this operation the walls or protection were simply the natural slopes of the underlying soil, no sheeting, sheet-piling, or other protection being used. The water was lowered progressively and held below the excavation requirements by a considerable number of well points, and the excavation was made by one high-power jet and two hydraulic dredges. This operation can be carried out successfully only in sandy soil which is free from a large percentage of clay or loam or large gravel.

Another interesting and unique type of braced wall cofferdam, also not entirely covered in any of the classes described, was that developed at the Staten Island end of the Narrows Siphon in New York City. In this a trench having a maximum depth of about 55 ft. and an approximate width of 19 ft. was sheet-piled, braced, and excavated in the bottom of the Narrows without unwatering. The walls were of steel sheet piling driven to form horizontal suspension members bellying some 2 ft. inwardly between master piles spaced about 15 ft. apart longitudinally and about 65 ft. long. As the excavation progressed by dredging, the 19-ft. trench between the master piles was braced by divers. It is interesting to note that when the grade was reached, a trench was built, also by divers, and the pipes for the siphon were loaded on cars at the land end and run into place and caulked by the same divers. U. S. Steel piling (12½ in., 38 lb.) was used between the master piles in this work. As this structure was not unwatered, and as the soil back of the trench was of firm sand which would tend to stand up well when not subject to erosion, it is not probable that there was sufficient pressure to heavily stress the jaws of the piling, although it was found in two or three cases the jaws had pulled apart at or near the bottom due to the heavy driving with a 7500-lb. McKiernan-Terry hammer through unusually hard soil including rip-rap and boulders.

In connection with the strength of the interlock or jaw-pull it is interesting to note the following from a pamphlet of the U. S. Steel Piling Co.:

With steel made in accordance with above specifications, average longitudinal strength per linear inch is as follows:

9¼-in. United States Steel Sheet Piling.......	5600 lb.
13¼-in. United States Steel Sheet Piling.......	9500 lb.

These values are taken at a yield point of the material and should be reduced from one-third to one-half to obtain safe working unit stresses. They can be increased by careful selection of sections at mill, also by additions to percentage of carbon in the steel. Without undue increase in carbon, it is possible to obtain 13¼-in. piling with interlock strength up to 12,000 lb. per lin. in., which is more than ample for any condition that has hitherto arisen.

The Black Rock and Havana Harbor cellular cofferdams were done under the direction of the U. S. Army Engineers, while the one with braced wall last described was done in 1914 under the direction of the Board of Water Supply of New York City.

OPEN CAISSONS

By J. C. Sanderson

An open caisson is a self-contained structure built on the surface of the ground and later sunk to position. During the process of sinking, the caisson has the function of a cofferdam.

Caissons are usually circular or rectangular in form. Large rectangular caissons are generally divided into compartments by cross walls.

The caisson may be a part of the permanent structure, as in bridge piers, or it may be the finished structure as in pump wells and mine shafts.

22. Use of the Open Caisson.—The open or dredging caisson has been used in bridge foundations for many years. More recently it has come into general favor for heavy building foundations, especially in wet ground. It can be sunk more rapidly and more economically than the pneumatic caisson, but can not always be used because of material from the outside running into the excavation.

23. Depth to Which Caisson May be Sunk.—Theoretically, the open caisson can be sunk to any depth. Practically, the depth to which a caisson may be sunk is limited by the weight required to overcome the skin-friction on the sides of the caisson. The greatest depth recorded for an open caisson is for a mine shaft in Germany, which was sunk 256 ft.

24. Amount of Skin-friction Developed.—The skin-friction developed varies with the character of the material penetrated, the amount of moisture present, the surface of the caisson walls, and the depth sunk. The unit intensity of skin-friction does not increase as rapidly with the depth sunk as might be expected. The passage of the lower part of the caisson smoothes the material and brings the moisture to the surface of the caisson, which acts as a lubricant. There is no method of calculating skin-friction and the amount that may be developed in each instance is a matter of experience and judgment. In general, skin-friction in the various materials varies in intensity in the following order: boulders and clay, gravelly clay, moist clay, gravel, sand, and silt.

The skin-friction for each material varies over a wide range and usually increases with the depth. For depths up to 100 ft., the skin-friction for sand may generally be taken at 250 to 500 lb. per sq. ft., gravel 300 to 600 lb., moist clay and gravelly clay 500 to 1000 lb. The above figures are not likely to be exceeded for average conditions. The skin-friction for quicksand is somewhat higher than for building sand. The skin-friction for so-called quicksand around New York City varies from 250 to 650 lb. per sq. ft. The run-in under the cutting edge is much larger for quicksand than for other materials. The skin-friction in boulders and clay may be very high.

Jacoby and Davis record a skin-friction of 1912 lb. per sq. ft. for the pivot pier of the Grand Trunk Bridge at Black Harbor on the Niagara River. The material was a very sticky red clay. The caisson was of concrete. The same authors give the maximum friction for one pier in the Cairo Bridge as 932 lb. per sq. ft. in sand.

The Foundation Company of New York City give the skin-friction for a concrete gun pit, sunk 130 ft. in clay and boulders at Washington, D. C., as 3500 lb. per sq. ft.

Friction as high as given in the above examples is seldom encountered.

25. General Design.—Caisson design is largely a matter of experience and judgment. The earth and water pressure, as determined by formula for pressure, is very easily provided for. However, in large and deep caissons that are pumped out, the stresses in the walls due to earth or water pressure may be high.

The unit stresses used in designing for loads to be carried after sinking should be very conservative. The process of sinking the

caisson may so rack and twist it that it has not the original calculated strength. The indeterminate stresses due to sinking are the ones most likely to be inadequately provided for. It is possible to get almost any conceivable condition of loading on a caisson during sinking, due to the variation of the skin-friction over the surface, the moisture, the hardness of the bottom, the tipping of the caisson, logs or boulders under the cutting edge, or the nature of the run-in of material under the cutting edge.

A caisson should be designed with a shearing strength equal to at least one-half its weight. A rectangular caisson should be assumed to be supported on diagonally opposite corners. In a caisson supported at the corners, torsion produces heavy bending stresses at wall intersections. The earth pressure is usually not balanced around the caisson; at such times, the caisson is acting as a beam or cantilever. In a caisson out of plumb, which has penetrated materials of different stiffnesses, this bending stress may be very high, especially if the material flows more readily under the cutting edge on one side than on the other. The lower portion of the caisson should be made especially strong to withstand pressure of material flowing under the cutting edge, extra load from tilting, impact from a sudden drop on logs or boulders, or shock from blasting. There should also be sufficient strength to prevent pulling in two, in case the top is friction bound.

The sides of the caisson should be straight. Sloping the sides to reduce friction makes the caisson more difficult to guide and may increase the friction by material falling into the crevices and jamming the caisson. The Chicago, Burlington & Quincy R. R., however, experienced but little difficulty in sinking in sand in the Platt River a pier which had a base 16 ft. square for 12 ft. above the cutting edge and a cylindrical shaft 11 ft. in diameter above the base. The general experience with tapered caissons would indicate this form would be much more likely to give trouble than a straight caisson.

26. Wooden Caissons.—Wooden caissons are built up of double walls of square 6- to 12-in. timbers with concrete between the walls to provide sinking weight. The caisson should be heavy enough to follow down as fast as the material is excavated. To get enough weight it is usually necessary to leave as much space as possible between the inside and outside walls, thereby reducing the dredging wells to a minimum. The dredging wells

should be 7 or 8 ft. square although caissons have been sunk with dredging wells as small as 5 ft. in diameter. Excavation is more expensive and slower with the small dredging wells, because such small buckets must be used.

The outside walls are built of square timbers drift-bolted together on 2- to 3-ft. centers. The inside walls are built of square timbers spaced 3- to 4-ft. centers. Large caissons are generally built of 12 × 12 timbers; small and shallow caissons may be made of 6 × 6, 8 × 8, or 10 × 10 timbers according to the size and depth. At the corners and intersections of cross walls the timbers may be halved, dove-tailed, or alternate sticks run through. The latter method is the cheapest and gives equally good results. The outside of the caisson and the dredging wells are sheathed with 2-in. surfaced plank laid vertically. Care should be taken to lay the sheathing smoothly to reduce friction and to avoid projections that might catch on obstructions.

The lower few feet of the caisson are laid solid in triangular form, bolted through horizontally from the outside to the dredging well and drift-bolted vertically. The lower edge, 8 to 12 in. wide, should be protected by a steel shoe.

27. Steel Caissons.—Double wall steel caissons are not frequently used at the present time on account of their greater cost. Their advantage over a wooden or concrete caisson is in their greater strength. The steel caisson should have the cutting edge stiffened and be braced sufficiently to prevent the plates from buckling.

28. Concrete Caissons.—Concrete is the most satisfactory material for caisson construction. It is heavy, economical, and with steel reinforcement has the required strength. The first consideration should be to get the walls heavy enough to sink without loading. The interior walls must be strong enough to transfer their load to the outer walls, when undermined, or to carry a considerable part of the weight of the outer walls in case of an unusually hard bottom under a part of the caisson. The caisson must be reinforced for bending in all directions and special care must be taken to make the corners and wall connections especially strong. When a caisson is racked out of plane, the cross walls produce a bending moment in the side walls instead of bracing them.

The cutting edge should be protected by a steel shoe and be heavily reinforced for some distance above the bottom.

29. Cutting Edges.—Figs. 16 to 20 inclusive show the usual forms of cutting edges. Figs. 16 and 17 are the forms most frequently used, and in general, the most satisfactory. Fig. 18 gives very little protection to the lower edge of the caisson. Figs. 19 and 20 lack stiffness and should not be used where obstructions may be encountered or when blasting may have to be resorted to. With the thin cutting edges, it is thought that the caisson follows the excavation more closely, thereby reducing the amount of material that runs in under the cutting edge.

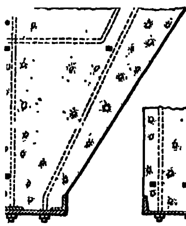


FIG. 16.

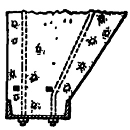


FIG. 17.

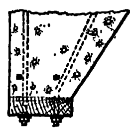


FIG. 18.

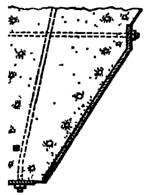


FIG. 19.

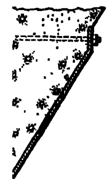


FIG. 20.

30. Landing the Caisson.—The site should be levelled off and cleared of all obstructions which would interfere with the free movement of the caisson. When the caisson is on land it is customary to dredge the site down 15 to 20 ft. before setting the cutting edge.

The cutting edge is set level on blocking. The blocking should not be longer than required to sustain the weight. The projection of the blocking on the outside of the caisson should be as short as possible. When ready to sink, all material except the blocking should be cleared away. As the dredging progresses, the blocking will be carried into the caisson by the material running in under the cutting edge.

Caissons in deep water are built on ways on the bank and floated to position. Guide piles are driven around the site so the caisson can be controlled until it is landed on the site. When the water is too shallow to float the caisson, it may be built on barges and floated to place. A frame work may be built up on the barges to support the caisson which is lowered by rods or by sinking the barges, or the caisson may be built directly on the barges and lowered by sinking the barges. As the caisson sinks, the barges are dragged from under the cutting edge. When the water is too shallow for a floating method, the caisson may be

built on a platform at the site. One or two rows of piles are driven each side of the site and capped. Cross-ties resting on the caps support the cutting edge. After completing the caisson, blocking is arranged on the platform and the caisson lowered to place by rods. In shallow water the site is sometimes dried up by a cofferdam.

31. Sinking Caissons.—Before starting dredging, the caisson should be built as high as practicable. The heavier the caisson the more closely it follows the excavation and the less material will run in under the cutting edge. A caisson that is narrow for its height may be unstable at the beginning and require bracing until it is deep enough into the ground to prevent overturning. In very soft ground a caisson may be difficult to control at the start. A caisson that is tilted can frequently be straightened by setting braces against the low side or by taking a heavy pull with a block and tackle.

Ordinarily, open caissons are sunk by dredging with a clam-shell or orange-peel dredge bucket, the clam-shell generally being the most satisfactory. The sand pump or ejector is sometimes used. Such a device was used very successfully and economically in sinking the piers in the sand of the Fraser River Bridge at New Westminster, British Columbia.

The excavation should be made as uniform as possible. If the caisson is out of plumb, the excavation on the high side should be carried ahead of the excavation on the low side. But excavating on the low side should not be entirely omitted as it is not practicable to plumb the caisson without lowering it as a whole. The general level of the bottom should be kept in a plane, tilting the plane as may be required to plumb the caisson.

The excavation may be but a short distance below the cutting edge or it may be as much as 10 to 15 ft. When the excavation is carried very far below the cutting edge, there is always the likelihood of a sudden drop, which may wreck the caisson. When the excavation is carried very far below the cutting edge and there is a sudden run of material under the cutting edge with an accompanying drop of the caisson, the material packs around the caisson, making it difficult to start the caisson again. In such cases it is generally advisable to load the caisson enough to get a uniform movement as the excavation is made.

Jet pipes built into the caisson walls have not proven very successful. They are generally plugged with mud and can not

be used when wanted. A jet free of the caisson and operated from the surface is much more satisfactory. The water jet should be used with caution. It is likely to cause a sudden drop in a deep dug caisson, and also has a tendency to cause the material to pack around, making it increasingly difficult to start the caisson after each application of the jet.

The usual built-in water jet discharges downward at an angle of about 45 deg. The water jet in the Bignell pile discharges through an elbow upward. It is claimed that this jet is very effective. It is quite probable, however, that the two forms of jets would be about equally effective for the same amount of water per square foot of friction surface.

The excavation should be carried on continuously. It is harder to start a caisson than it is to keep it moving. Caissons to be landed on rock stop a short distance above the rock. They usually can be brought to a fair bearing by pumping them down rapidly and washing the material from under the cutting edge.

When the caisson is not heavy enough to overcome the skin-friction, it is loaded with bags of sand, railroad rails or cast weights of concrete or iron. Loading a caisson slows up the sinking and materially increases the cost of the work. Blasting may be resorted to on especially stubborn caissons, where the material is very hard to dig, or the boulders and logs hard to break up. The usual method is to explode from one-half to a stick of dynamite inside the caisson. The Foundation Company reports a very remarkable case of blasting a caisson down in boulder clay. After the bottom had been dug 10 to 15 ft. below the cutting edge, a series of holes were drilled about 40 ft. deep all the way around the caisson and about 30 ft. back from it. Each hole was loaded with a stick of dynamite and all were fired at the same time. A few minutes after each shot the caisson would sink 6 to 8 ft. The caisson was sunk by the above method about 90 ft.

The open caisson can not be as easily controlled as a pneumatic caisson nor as accurately located. Generally the larger the caisson, the easier it is to keep plumb and the more accurately it can be landed. To keep small caissons plumb it is usually necessary to use shores.

32. Sealing Caissons.—When the caisson has been sunk to the required grade, the bottom should be dredged to a uniform surface. If the bottom is under water, as is usually the case,

careful soundings should be taken to see that the surface slopes uniformly from center of the dredging well to the cutting edge. A diver should be put down to clean the material from under the steps at the cutting edge. The concrete may be deposited by either a tremmie or bottom dump bucket. The tremmie is usually heavy and difficult to handle. The bottom dump bucket is more easily handled and for deep work will give the best results. The concrete must be poured continuously and carried through the full thickness to avoid laminations.

The sealing concrete is not generally reinforced, as it is not practicable in most cases to reinforce it. It should not be leaner than a 1 : 2 : 4 mix, as concrete poured through water is not likely to be as good as concrete poured in the dry. Concrete poured through water has, in addition to a deposit of mud and laitance, a layer of shelly concrete on top which may be from a few inches to a foot thick. There should always be a reduction in the unit stresses for concrete poured through water. The depth of seal for small dredging wells varies usually from a thickness equal to the smallest dimension of the well to a thickness of one-half that amount. If the above rule were followed in large dredging wells, it would be quite expensive, consequently the thickness of seal is reduced to one-fourth the least dimension of the well, or even less. When the dredging well is round or nearly square the seal may be figured as a flat slab supported at the edges. At the completion of the excavation the bottom is more or less in the form of a dome. If there is strength enough in the cutting edge to carry the ring tension, the seal may be figured most economically as a dome.

33. Rate of Sinking.—When the caisson is heavy enough to overcome the skin-friction, the rate of sinking is dependent upon the amount of material that can be excavated. The amount of material excavated exceeds the displacement of the caisson. In sand and gravel with a light caisson the run-in may amount to 50 per cent of the displacement. With a heavy caisson the run-in should not be over 25 per cent. In quicksand with a light caisson the run-in may be over 100 per cent of the displacement. The run-in for clay is usually negligible.

34. Cost of Sinking.—The cost of excavation is practically the only indeterminate variable for a caisson that is heavy enough to sink by its own weight.

The cost of excavation varies with the location of the work, the character of material to be excavated, the size and depth of the caisson, the size of the dredging wells, and the method of disposing of excavated material. At current wages, excavation in large wells in sand or loose gravel should cost from 60 to 80 cts. per cu. yd. Excavation in the same material in 5-ft. dredging wells should cost from $1 to $1.50 per cu. yd. The cost of removing logs that may be encountered is not included in the above prices. The cost of dredging increases with the depth of water through which the dredging is done.

The increase in cost is due to reduction in speed, to material being washed out of the bucket, to difficulty in placing the bucket, and to the weight of the water compacting the material. The amount of material washed out by the water is a large item after orange-peel buckets have been in service for some time.

PNEUMATIC CAISSONS

By J. C. Sanderson

Pneumatic caissons have been in general use in Europe and this country for many years. The pneumatic caisson may be roughly described as a rigid box with the bottom omitted. It is kept free of water by compressed air.

A cofferdam, extending above the water line, is attached to the top of the caisson. As rapidly as the caisson is sunk the masonry is built, keeping the top above the water. The cofferdam is frequently utilized as a form for the concrete work of the lower part of the pier.

35. Depth Limitation.—The pneumatic caisson method is limited practically to a depth of not over 110 ft. below water level or an air pressure of 50 lb. per sq. in. There are a few instances where pneumatic caissons have been sunk a few feet deeper than 110 ft. and have had an air pressure of a few pounds above 50 lb. to the sq. in., but a pressure of about 50 lb. per sq. in. is the recognized limit of endurance for the human system.

The cost of pneumatic work increases very rapidly as the air pressure increases on account of the shorter shifts the men can work and the longer rest periods required. There is also a decided increase in the number of cases of caisson disease when the pressure reaches 40 to 50 lb. per sq. in.

36. Skin-friction Developed.—The skin-friction for pneumatic caissons is not quite as high as for open caissons because the compressed air, as it bubbles up along the side of the caisson, acts as a lubricant.

37. General Design.—All of the statements in regard to stresses under open caissons apply with equal force to pneumatic caissons. In the pneumatic caisson, however, the pier above the working chamber is usually solid masonry, except for the comparatively small shafts left for access to the working chamber. The walls of the working chamber are subjected to stresses of the same character and intensity as the lower part of an open caisson. The walls of the working chamber must be designed for the increased load of a caisson out of plumb, unbalanced lateral pressure, the pressure of material flowing under the cutting edge, and the impact when striking logs or boulders.

In the earlier designs the roofs of caissons were made of very heavy timber construction. In the more recent designs the roof timbers are made only strong enough to carry 4 to 6 ft. of wet concrete. The roof should be stiff and be sufficiently well anchored to the side walls so the caisson will keep its form while being landed on the site. The roof should be strong enough to transmit the weight of the lower part of the pier to the cutting edges. The roof concrete should be reinforced and allowed to fully set before sinking is started. After the concrete of the pier has set, it will be self-sustaining. The air pressure can not be relied upon to carry a part of the load as the pressure may be reduced quickly to zero for short intervals. The working chamber should be made as near air tight as possible, and should be from 6 to 7 ft. high. Higher walls afford more working space when the caisson is partially filled with sand. While the higher walls make the excavating more economical, they increase the cost and difficulty of building the caisson.

38. Wooden Caissons.—There have been three general designs for the side walls of wooden caissons. The earlier designs, originated in the days of cheap lumber, had V-shaped side walls laid up of solid timbers drift-bolted together. Alternate timbers were extended through the course instead of halved. An adaptation of the above design is to build the outside face of the wall of a single course of timber either 6 × 12 or 12 × 12, and a single course of timber for the 45-deg. sloping wall. The space between the walls is filled with reinforced concrete. The straight

walled caisson is built of two courses of 12 × 12 horizontally, drift-bolted together vertically, and bolted through and through horizontally, alternate timbers being cut at the corners.

The straight walled caisson may also be made of one or two horizontal courses and a vertical course. The outside course is usually made of 12 × 16's and extended several feet above the roof. The roof timbers are gained into the vertical timbers to transmit the roof load to the caisson walls. The above sizes of timber are for large and deep caissons. For small and shallow caissons the size of the timbers may be reduced.

The walls of the straight walled caisson should be braced at intervals of about 10 ft. by struts and rods extending across the caisson. The V-shaped walls require much less bracing than the straight wall. The straight caisson wall is more frequently used than the V-shaped because it is less difficult to excavate under the cutting edge. The lower course of timber should be protected by a steel shoe. The roof must be stiff enough to keep the caisson in shape and carry the imposed loads. If the imposed load is concrete, as is usually the case, it is necessary to carry only the first pour of concrete until it sets. Stone masonry would require a considerably stronger roof. The roof must be strong enough to carry the air pressure in cases where the superimposed load is concentrated on the caisson walls.

In large caissons the wall bracing may take the form of bulkheads with tie and truss rods. In such cases the bulkheads with trusses may be used as a support for the roof.

The entire inside of the working chamber and the outside of the walls should be sheathed with 2- or 3-in. sheathing. The sheathing should be surfaced on one face and planed on the edges for caulking or should be tongued or grooved. To get a uniform surface, all caisson timbers should be surfaced on one side and one edge.

The cofferdam, built from the top of the caisson to a few feet above water line, must carry the water load until the masonry is put in. When concrete is used it is customary to utilize the cofferdam as a form. When the pier is of stone masonry the cofferdam is braced to the pier or the space between the pier and cofferdam is filled with sand.

39. Concrete Caissons.—Concrete caissons for bridge piers have not been used to any great extent in this country. On account of their weight they are difficult to build and to land on

the pier site. Also, it is necessary to allow a considerable time for hardening before the sinking is started. A large, comparatively shallow caisson of wood will stand much more abuse than a concrete one, even though the latter has had months to set. Concrete is a good material for a caisson that can be built on the site and a considerable depth of pier added before sinking. Under such conditions concrete caissons are very extensively used around New York for building foundations.

40. Steel Caissons.—On account of their cost, steel caissons are not used to any great extent in this country except for small building piers around New York City. In a vertical walled steel caisson the side walls are built of plates and angles, knee-braced to the roof beams. The knee-braces are spaced 4 or 5 ft. on centers. The V-shaped walled caisson has a vertical and inclined wall of plates and angles with diaphragms at intervals of 6 to 8 ft. The space between the walls is filled with concrete. The roof is made of beams or girders with a plate riveted to the bottom flange. The cutting edge should be reinforced for stiffness.

41. Cutting Edges.—The cutting edge best adapted to pneumatic caissons is that shown in Figs. 19 and 20, p. 118. The sharp edges permit the material to be dug out close to the side wall without allowing a great quantity of air to escape.

42. Sinking Caissons.—The site should be cleared of mud and levelled. In swift streams with soft bottoms the bottom on the up-stream side of the caisson is sometimes paved with sand bags to prevent scouring and undermining the caisson before the sinking is well started.

The best and most economical method of sinking a pneumatic caisson is to have just enough weight to keep the caisson moving as fast as the material is excavated from under the cutting edge. The excavation should be carried on continuously because after a caisson has stopped, there is always difficulty in again starting it.

It is usual to excavate the material about a foot below the cutting edge, except close to the cutting edge. In clay the excavation can be carried somewhat deeper. The material under the cutting edge is then removed, and the air pressure reduced enough to let the caisson settle to the bottom of the excavation. If the caisson is much heavier than required to overcome friction the working chamber may be practically filled with mate-

rial making it difficult for the men to clear away a space large enough to work in. When passing through hard material or boulders, it is important to see that the excavation is made amply wide so that the caisson will not jam. The excavated material is removed by buckets working through air locks or by a blow-out pipe.

The blow-out pipe is simply an iron pipe about 5 in. in diameter extending from the deck to the working chamber. There is an elbow at the top and a piece of flexible hose with a flap valve at the bottom. The excavated material is shovelled around the end of the hose and the valve opened. All of the material in front of the valve is very quickly blown out. The process is so rapid that the valve need be opened but a short time. Material is not generally blown out until the air reaches a pressure of about 20 lb. per sq. in. The abrasion of the sand on the elbow of the blowpipe wears it out very rapidly.

Clay and rocks are best removed by a bucket. The air locks now in use permit the bucket being taken out of the caisson without disconnecting the bucket from the rope.

As the caisson is sunk it is rarely quite plumb nor is it exactly in the correct location. The caisson can usually be made to move in the direction desired, provided the material penetrated is not extremely hard. To move the caisson laterally the side of the caisson opposite the direction to be moved should be undercut and the other side banked. The caisson is then allowed to sink out of plumb. The cutting and banking is then reversed and the caisson brought to a vertical position. The caisson can not be moved laterally without sinking it at the same time.

One of the land piers of the Missouri River Bridge at Omaha was moved 5 ft. laterally in sinking 8 or 10 ft. In addition to the cutting and banking, a heavy pull was put on the pier by a block and tackle. Inclined shores also were set in the working chamber to assist in the movement.

43. Sealing Caissons.—If the caisson when sunk to grade is in sand, gravel, or hard pan, the bottom should be levelled and the material dug from under the cutting edge. If the caisson lands on sloping rock overlaid by coarse sand or gravel, it is usually better to excavate the rock and continue the sinking until the cutting edge reaches the low point of rock. If the sloping rock is overlaid by fine compact sand or clay, the excavation may be carried below the cutting edge to rock, by banking the sides of

the excavation with bags of sand plastered over with clay, or the excavation may be carried to rock by jacking sheeting down. The risk of excavating below the cutting edge increases as the air pressure increases. All loose rock should be cleaned off, all rotten rock removed, and all crevices cleaned out. Sloping rock should be stepped to give horizontal surfaces. Filling the working chamber with concrete is a very important part of the finishing work on a caisson and requires a great deal of care. It is customary to fill in concrete to 2 or 3 ft. above the cutting edge, and then build the concrete up in steps to within 6 in. of the roof by bulkheading. The last 6 in. is filled by ramming in comparatively dry concrete. This process is slow, tedious, and expensive. Another method is to fill the working chamber to within a foot of the roof with medium wet concrete. After the concrete has had time to set, the air pressure is released and the remaining 12 in. are filled with concrete wet enough to flow. The air is forced out at 2-in. air pipes placed near the corners of the caisson.

44. Rate of Sinking.—The rate of sinking pneumatic caisson depends somewhat on the amount of skin-friction, difficulty with logs and boulders, etc., but primarily, it depends on the rate at which the excavation can be made. For caissons of usual proportions a rate of 1½ ft. per day is considered a fair average.

45. Cost of Excavation.—The cost of excavation in pneumatic caissons at existing wages is \$4 to \$5 per cu. yd. for average material. The above figure does not include anything to cover cost of cleaning away obstructions.

46. Caisson Diseases.—As the air pressure in the caisson is increased, the fluids of the body absorb increasing quantities of air in accordance with Dalton's Law of Solution of Gases in Fluids; which is, the amount of gas dissolved in a fluid is proportional to the pressure of the gas surrounding the fluid.

When the air pressure is lowered too rapidly, the absorbed gases are thrown out of solution more rapidly than the body can eliminate them and bubbles are formed in the blood, tissues, or joints. The cure is to put the patient under pressure and decompress more slowly. Most states have laws regulating the rate of decompressing and the length of shift, which vary according to the intensity of pressure. They also require that a hospital compression tank be provided on the work for the treatment of caisson diseases.

47. Cofferdam Caissons.—Both dredging and pneumatic caissons are used as cofferdam caissons. In city work the dredging caisson is not used very frequently on account of the ground level outside of the caisson being lowered by the material running under the cutting edge of the caisson.

Caissons, when used to make a cofferdam, are sunk close together, with a key between, which is sealed after the caissons are in place. In some instances the keyway is excavated by hand, the space between the caissons being plugged as the excavating proceeds. If water is encountered, it is forced out with compressed air. The keyway may also be cleaned out with a small orange-peel bucket, blown out with an air jet, or with a water jet. All of the above methods have been successfully used. Cofferdam caissons are narrow for their length and height and require care in guiding to keep them plumb.

CAISSON DETAILS

By J. C. Sanderson

Example 1.—Fig. "A" shows details of one of the boiler house piers of the Kansas City Power and Light Company's Power Stations on the Missouri River at Kansas City, Missouri.

The seal and cap were of 1:2:4 concrete; the walls of 1:2½:5 concrete reinforced as shown. The space between the seal and cap was left empty. The pier was designed for a bearing capacity of 6 tons per sq. ft., exclusive of the weight of the pier. The material penetrated was fine sand. Numerous logs were excavated in sinking the piers on this job, causing considerable trouble and expense. The skin-friction was 300 to 400 lb. per sq. ft.

The water tunnels for this power station were sunk in sections—16 ft. wide, 29 ft. deep, and 43 ft. long—as open dredging caissons. The sections were sunk 1 ft. apart. The space between caissons was closed on the outside by driving sheet piling, on the inside by bracing a form against the concrete, the sand cleaned out, and the opening filled with concrete.

The seal was cast through the water; the top which was above low water was cast in the dry.

Example 2.—Fig. "B" shows details of an open dredging caisson sunk on the Ohio River for the Union Gas & Electric Co. of Cincinnati, Ohio, for a condenser pit. The caisson was 68 ft. inside diameter with wall 8 ft. thick. The requirements of sink-

ing weight determined the wall thickness. The caisson was sunk 60 ft. by open dredging through a cinder fill containing stone, clay, refuse from a gas plant, piles, etc., and 20 ft. into river

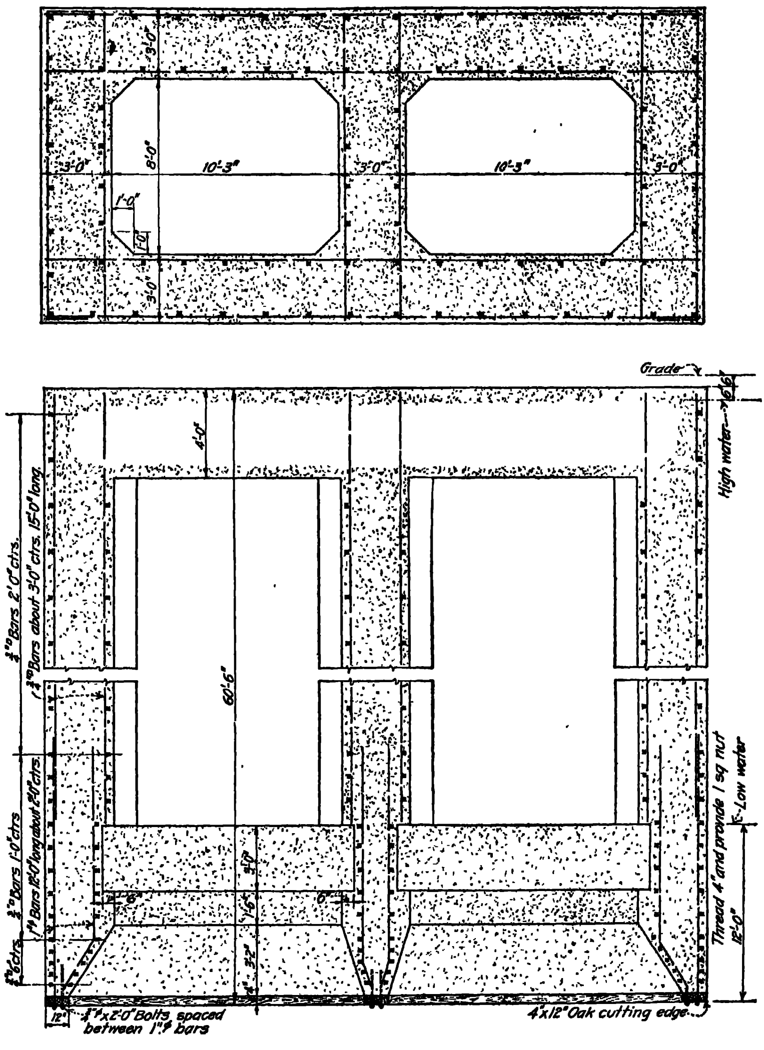


Fig. A.

gravel. The caisson was designed to resist a 70-ft. head of water. The walls were of 1:2½:5 concrete reinforced both ways with

1-in. bars spaced 2 ft. centers on the inside and outside of the wall.

The seal was 1:2:4 concrete cast by bottom dump bucket through about 30 ft. of water. The seal was in the form of a section of a sphere and figured as a dome without reinforcing. The

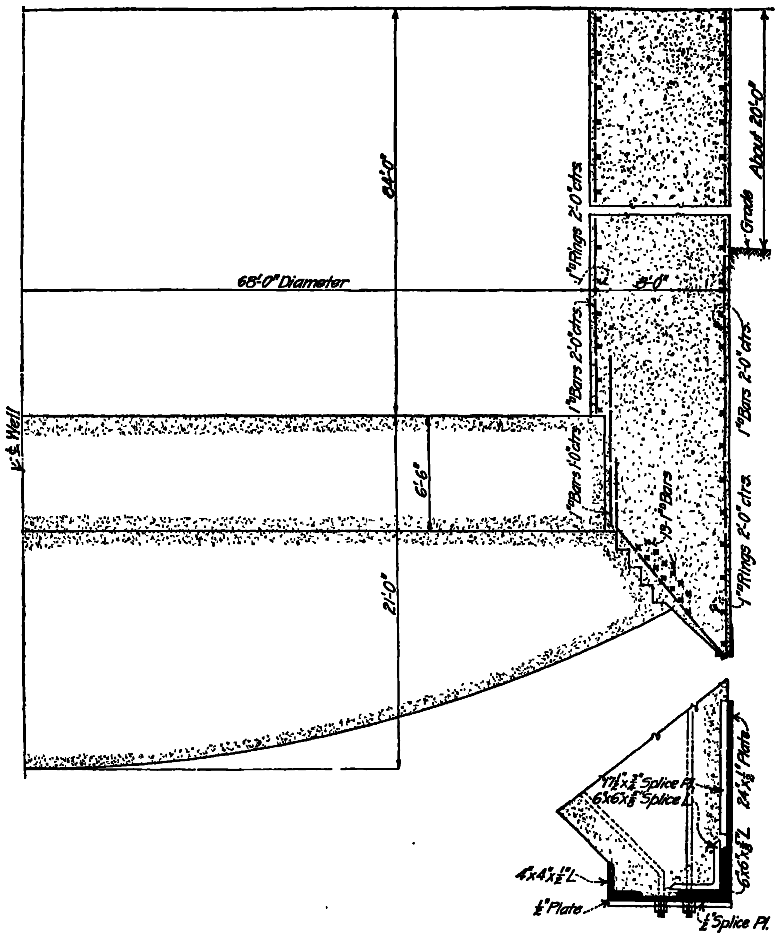


FIG. *B*.

side walls were reinforced for ring tension. After the caisson was pumped out a finished bottom 6½ ft. thick containing the intake and discharge water tunnels was installed. Before the finished bottom was installed the sealing plug carried a 50-ft. head of water

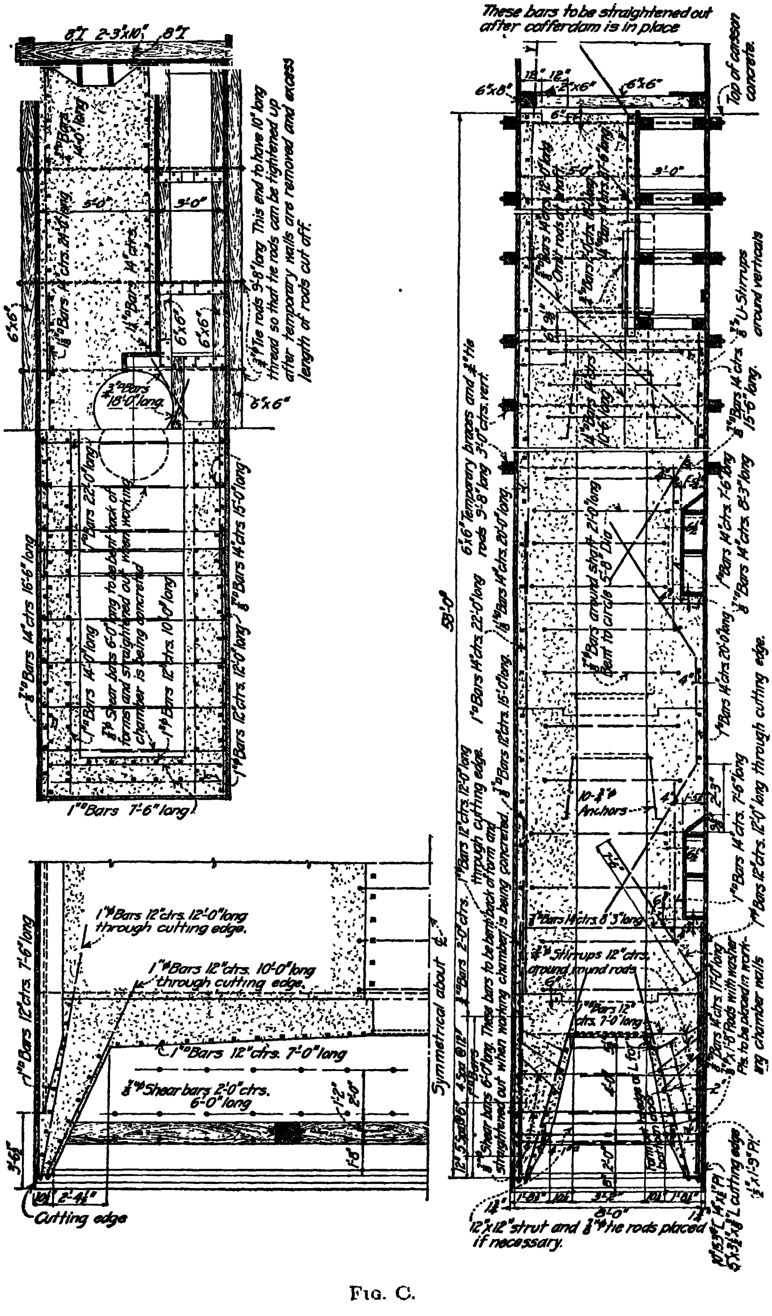

These bars to be straightened out after cofferdam is in place
Top of caisson concrete.
1"⌀ Bars 7'-6" long
1"⌀ Bars 12" ctrs. 12'-0" long through cutting edge.
1"⌀ Bars 12" ctrs. 10'-0" long through cutting edge.
1"⌀ Bars 12" ctrs. 7'-0" long
Cutting edge
12"x12" strut and ⅞"⌀ tie rods placed if necessary.
10-⅞"⌀ Anchors
Symmetrical about ℄
58'-0"
8'-0"

Fig. C.

with practically no leaks. The caisson was built up in 7-ft. lifts as the sinking progressed. The weight of the caisson was just enough to overcome the skin friction which amounted to about 1000 lb. per sq. ft. The caisson was landed 4 in. off center and 2 in. out of vertical.

Example 3.—Fig. "C" shows details of a pneumatic wall caisson for the Federal Reserve Bank at New York. On account of the excessive run-in under the cutting edge when sinking piers by the open dredging method, it is necessary in cities to use the pneumatic caisson. The caissons are sunk in contact. After sinking, the hexagonal opening between caissons is cleaned out and filled with concrete, this making a water-tight retaining wall around the building site.

After the caissons are sunk and sealed the basement excavation is made, the permanent struts being installed as the excavation progresses.

Example 4.—Fig. "D" shows details of Pier No. 4 of the McKinley Bridge across the Mississippi River at St. Louis. The caisson was of timber and concrete construction. The walls were constructed of 6 × 12-in. timbers laid edgewise. The cross struts are of 6 × 8 and 10 × 10 timbers spaced 11 and 12 ft. horizontally and 3 to 4 ft. vertically. The wall timbers were laid with intermediate butt joints and halved corner joints and are drift-bolted together with 7/8-in. drifts, 2 ft. 8 in. long, spaced 5 ft. on centers in every course. Posts, 6 × 8-in., were put at intersection of struts with the side wall and 3/4-in. tension rods were used under each brace. The side walls were stiffened by 4 × 8-in. diagonals spiked on the inside. The working chamber was 7 ft. high with the sides and ends sloped at 45 deg. The roof of the working chamber was made of a single course of 12 × 12-in. timbers. Alternate timbers were halved into the side walls; the other roof timbers were stopped at the 45-deg. slope timbers which were 6 × 12-in. laid solid. The walls of the working chamber were braced by one longitudinal and five transverse struts extending from wall to wall. There were two 1 1/4-in. rods with turn-buckles at each strut. The outside of the caisson was sheathed with 2-in. plank, the inside with 3-in. All sheathing was thoroughly caulked and all joints in ironplates coated with asphaltum.

The cutting edge consisted of two 3/8-in. plates outside, 8 in. and 33 in. wide, and a 36-in. plate on the inside bent to fit the

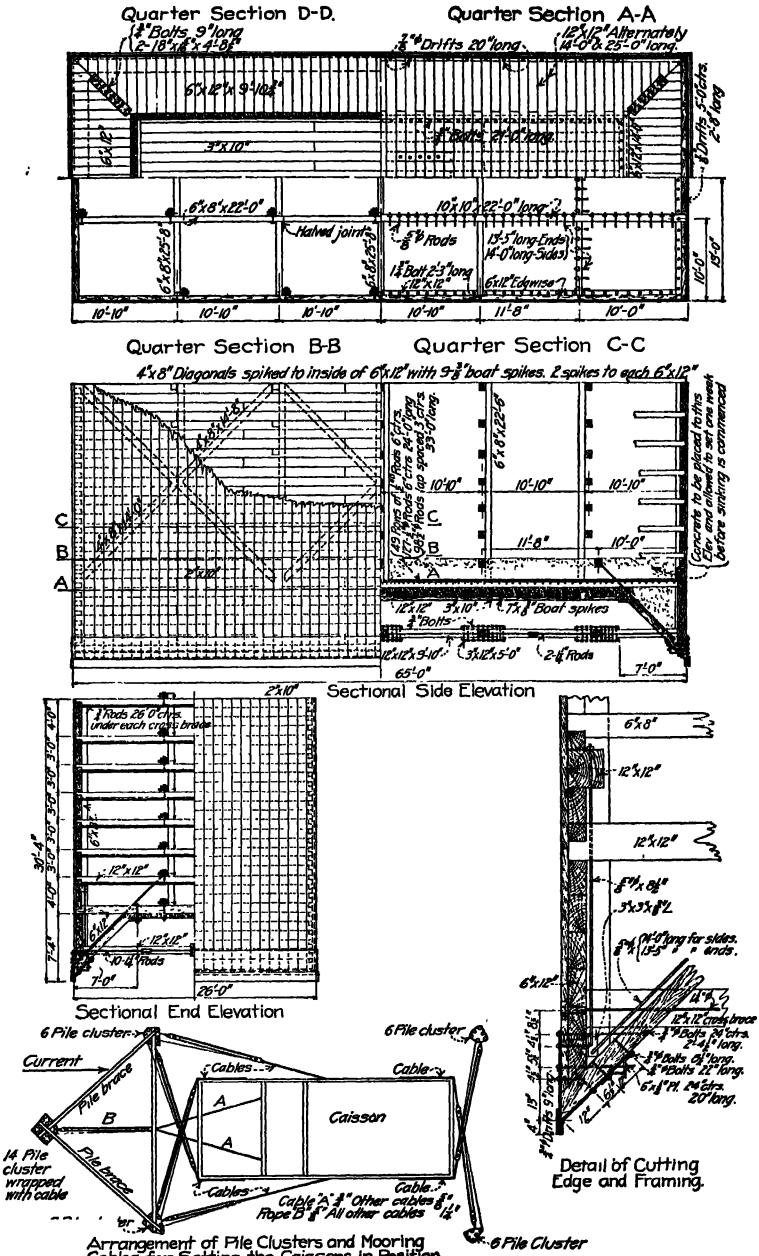

Quarter Section D-D.
Quarter Section A-A
Quarter Section B-B
Quarter Section C-C
4"x8" Diagonals spiked to inside of 6"x12" with 9⅜" boat spikes. 2 spikes to each 6"x12"
Sectional Side Elevation
Sectional End Elevation
Detail of Cutting Edge and Framing.
Arrangement of Pile Clusters and Mooring Cables for Setting the Caissons in Position
6 Pile cluster
Current
Pile brace
Caisson
14 Pile cluster wrapped with cable
Cables
Cable
6 Pile Cluster
Halved joint
10'-10"
11'-8"
10'-0"
65'-0"
26'-0"
7'-0"

FIG. D.

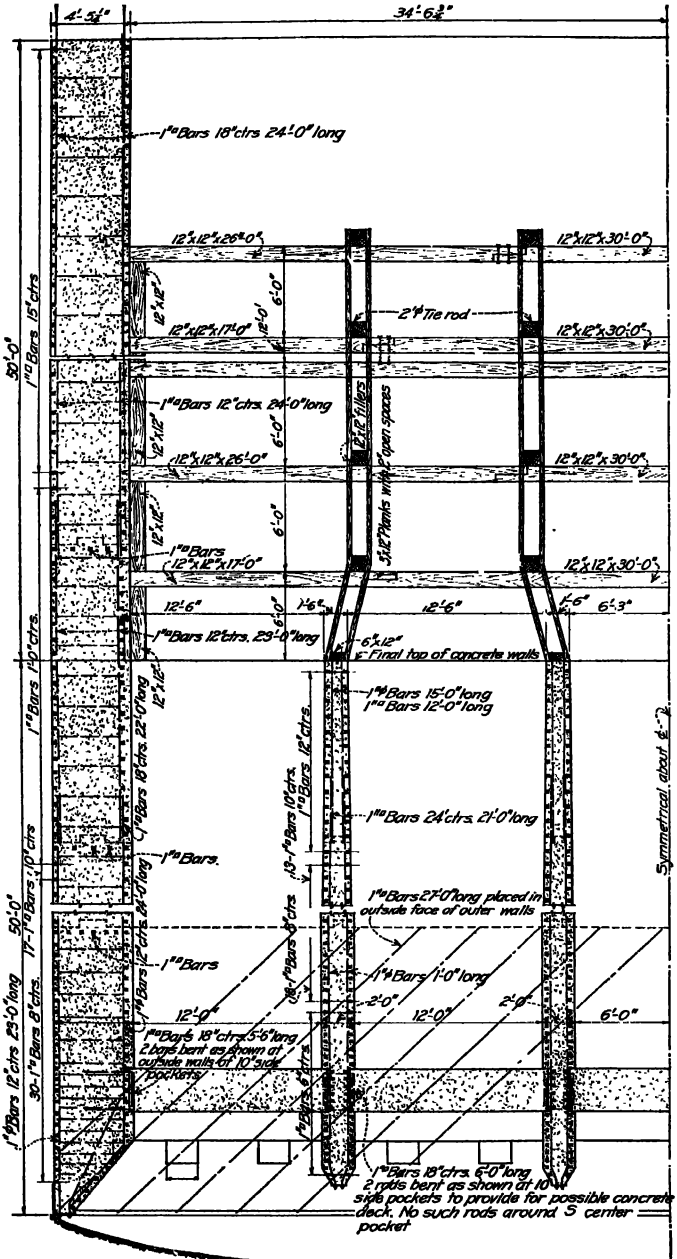

4'-5½"
34'-6¾"
1"□ Bars 18" ctrs 24'-0" long
12"x12"x26'-0"
12"x12"x30'-0"
2"ϕ Tie rod
12"x12"x17'-0"
1"□ Bars 12" ctrs. 24'-0" long
12"x12"x26'-0"
1"□ Bars
12"x12"x17'-0"
12'-6"
12'-6"
6'-3"
1"□ Bars 12" ctrs. 23'-0" long
6"x12"
Final top of concrete walls
1"ϕ Bars 15'-0" long
1"□ Bars 12'-0" long
1"□ Bars 24" ctrs. 21'-0" long
1"□ Bars.
1"□ Bars 27'-0" long placed in outside face of outer walls
1"□ Bars
1"ϕ Bars 1'-0" long
12'-0"
2'-0"
6'-0"
1"□ Bars 18" ctrs. 5'-6" long 2 bars bent as shown at outside walls of 10" side pockets
1"□ Bars 18" ctrs. 6'-0" long 2 rods bent as shown at 10 side pockets to provide for possible concrete deck. No such rods around 5 center pocket
50'-0"
1"□ Bars 15" ctrs
1"□ Bars 1'-0" ctrs.
17-1"□ Bars 10" ctrs
30-1"□ Bars 8" ctrs.
1"ϕ Bars 12" ctrs 23'-0" long
Symmetrical about ℄

Fig. E.

45-deg. slope. The steel plates extended 4 in. below the caisson timbers. Lug plates were bolted to the inside timbers of the cutting edge to take the weight during construction.

The concrete in the V-shaped cutting edge was reinforced with ⅝-in. rods spaced 1 ft. on centers along both the vertical and inclined walls. These ⅝-in. rods were anchored to the lugs on the cutting edge.

The concrete on the timber roof was reinforced with two layers of ½-in. round rods in each direction spaced 6 in. on centers.

The caisson was built on shore on landing ways and landed in position by guide piles shown.

The material penetrated was mainly coarse sand, quick sand, gravel, and small boulders.

Excavated material was blown out by air pressure.

The skin-friction was from 300 to 600 lb. per sq. ft. at the starting of the caisson but after the caisson was started the friction was considerably less.

Example 5.—Figs. "E" and "F" are plans and sections of a reinforced concrete gun pit sunk near Washington, D. C. for the Government by the Foundation Company of New York City.

Borings taken on the site of this caisson showed a comparatively stiff clay for a depth of 130 ft.

The ground was excavated to a depth of 20 ft. as an open cut before the cutting edge was set. The sinking progressed satisfactorily for the first 20 ft. when a layer of boulders 15 ft. thick was struck. The caisson which had been designed for a friction of 500 lb. per sq. ft. would not penetrate the boulder strata when loaded to overcome a friction of 3500 lb. per sq. ft. The caisson was dynamited by drilling a series of holes into the boulder strata 10 ft. apart and 30 ft. away from the caisson all the way around. Each hole was loaded with a stick of dynamite and all discharged at once. A minute or two after the explosion, the caisson would drop suddenly 6 or 8 ft. By repeating this process the caisson was sunk 90 ft. to the required depth. The caisson was landed without injury 1 ft. out of plumb. The seal was placed in the dry by pumping out the caisson.

Example 6.—Fig. "G" shows details of the caisson for the draw rest pier of the Chicago & North Western R. R. bridge at Pierre, South Dakota. The caisson was of timber construction. The walls were constructed of an outside course of 12 × 16-in. timbers laid vertically and two courses of 12 × 12-in. timbers laid

horizontally. The vertical timbers extended to the top of the roof and were notched 6 in. into the roof for bearing. Alternate timbers of the horizontal timbers were run through at the corners. The roof was constructed of two courses of 12 × 18-in.

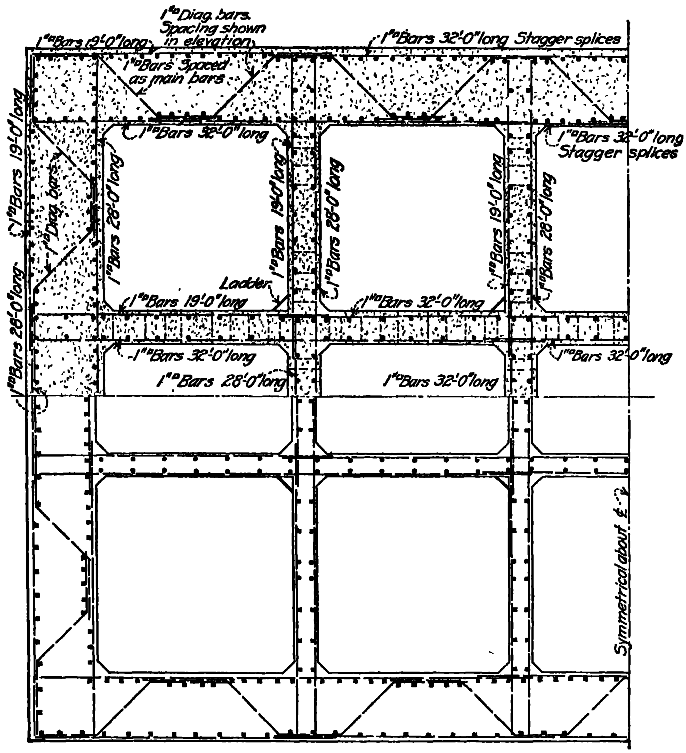


Fig. *F*.

timbers laid transversely and two courses of 12 × 12-in. timbers laid longitudinally. The cross struts were 6 × 12-in. timbers at the top and 12 × 12-in. at the bottom. The bottom timbers were halved at intersections. At the intersection of struts and at the intersection within the wall, 12 × 12-in. posts were put between the top and bottom timbers. There were two 2-in. rods with turn buckles at the bottom chord of each strut. There were three transverse struts and one longitudinal one.

The cofferdam was laid solid of 12 × 12-in. timbers and braced by three lines of 12 × 12-in. timbers laid transversely and two lines longitudinally. Bracing timbers were spaced on 4-ft.

centers vertically, halved into the side walls and drift bolted at intersections.

All timbers were drift bolted together with $\frac{7}{8}$-in. drift bolts

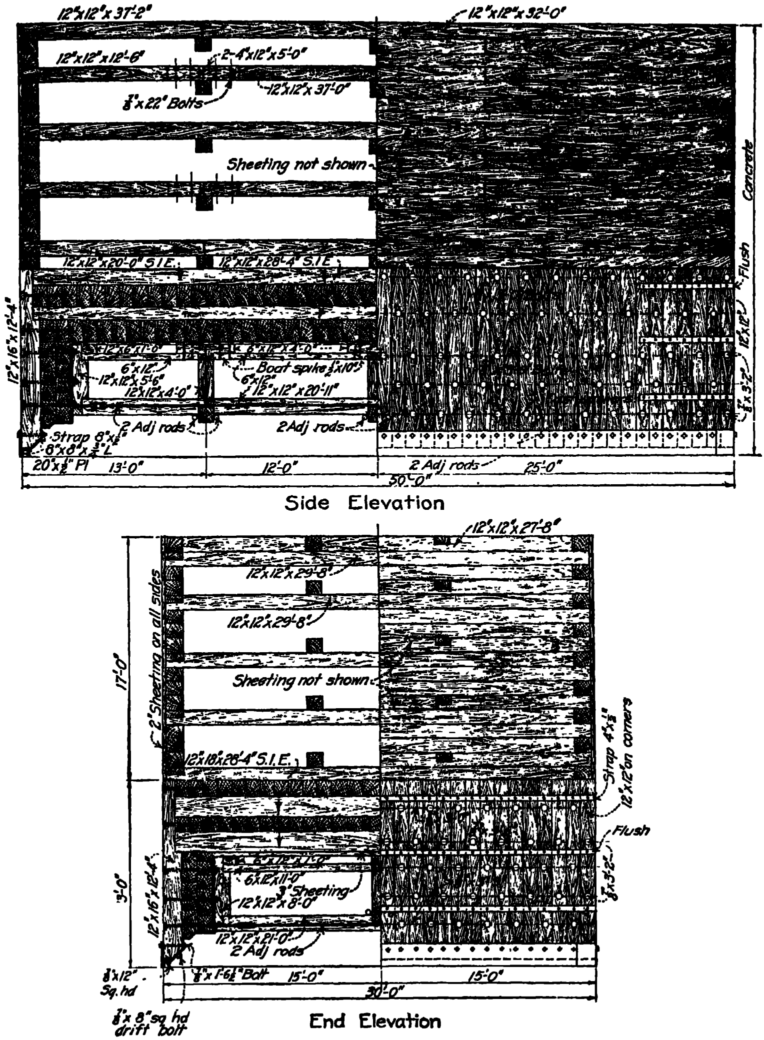


FIG. G.

the full depth of the timbers drift bolted together and spaced 4-ft. on centers along each timber. The outside of the caisson was sheathed with 2-in. plank, the inside with 3-in. All sheathing was

thoroughly caulked. All timbers were sized to dimension. The cutting edge consisted of an 8 × 8 × ¾-in. angle and a 20 ×½-in. plate. The horizontal leg of the angle was fastened by 8 × 12-in. plates spaced on 1-ft. centers. The caisson was sunk through 40 ft. of sand and gravel, and 5 ft. into grey clay shale.

TIMBER PILES

By Walter Cahill

48. "Pile" Defined.—The word pile is derived from the Anglo-Saxon pil, meaning an arrow or sharp stake. Also from the Latin pilum, meaning javelin, and from the Latin pila, meaning a pillar.

In an engineering sense a pile may be defined as an element of construction composed of timber, concrete, or iron, or a combination of these that is either set, driven, or screwed in the ground vertically, or nearly so, for the purpose of increasing the power of the pile to sustain a weight which is to rest upon it, or for resisting a lateral force.

49. Earlier Uses of Piles.—Historically considered, archæologists and ethnologists recognize that piles were used in Europe by prehistoric races to form rude foundations for supporting their dwellings in lakes, a short distance from shore, and generally in shallow water. Whole villages were built this way. Some piles were driven and some were merely set with stone placed around them afterwards to compact the ground. These piles were of wood 2½ to 10 in. thick, and up to 25 ft. long. Their relics are common in the lakes of Switzerland. On account of its manifest protective features, this type of so-called lake or lacustrine dwelling is still used by many savage tribes in different parts of the world.

Cæsar's Commentaries describe pile bridges built by his soldiers previous to the Christian Era.

In medieval ages Venice and the Holland cities used wooden piles for foundations, these being driven by mallets or by metal hammers lifted by men or by animals; also by wiggling or rocking to and fro into place in soft ground, without driving, the trunks of small trees, as is still done by farmers in northern Europe to establish platforms for drying hay, etc.

Timber piles have been used in Europe continuously since medieval times, the trunks of trees from their forested districts

naturally suggesting themselves for the purpose. Iron shoes or points were later employed. Metal piles were first used about the middle of the Nineteenth Century, mainly in England, in the form of cast-iron screw-piles, and disk piles to provide large bearing area. Their use has been very limited, such as for lighthouses, ocean piers, etc. Concrete piles were introduced about the year 1900 when the American Portland cement industry became active. Previous to that time caissons or piles of sand 3 to 4 ft. in diameter were used to support bridge foundations. The sand was poured into a vertical cavity prepared for the purpose, and served chiefly to compact the earth, and thereby increase its bearing power.

50. Pile Foundations in Modern Construction—General Features of Design and Construction.—In modern engineering construction pile foundations are employed where the soil is manifestly improper for spread foundations; for example, where the ground is too soft to support the load without compressing or compacting, or where it is desired to support the load on hardpan or rock which is overlaid by very soft or fluid material.

In driving piles into hardpan, it is essential not to drive through the hard stratum and into soft material which may be underneath. Some Chicago conditions are cases in point.

Pile foundations, in general, consist of a base of timber, steel, stone, brick, or concrete masonry, or a combination of these, supported by piles which distribute the load of the structure resting upon it, through a considerable depth, to the earth, hardpan, or rock below.

In cases where the pile is employed to consolidate soft ground, the so-called pile action occurs, and the limiting condition of load is either the adhesion of the ground to the surface of the pile—namely the friction—or the compressive resistance of the material in the pile itself. The tip of the wooden pile in such cases may be as small as 6 in. Where a pile is used to go through soft material to hardpan or rock, it receives but little lateral support from the ground, and consequently acts principally as a column; hence it should have a larger tip than if its resistance depends mainly on friction. Nine- to ten-inch tip is desirable, though somewhat difficult to obtain in the ordinary market.

In either case, the pile so employed is called a bearing pile, which is a general term applying to any pile which carries a

superimposed load. Examples of bearing piles are found in foundations of bridges and buildings, elevator foundations, power plants, and ore and coal storage docks. In the cases of storage docks and power plants on river banks, and particularly where compressible material rests on a harder stratum which slopes toward the boat channel, wooden piles are often driven closely together over large areas and well into the hard stratum to prevent disastrous slips into the water later, when the dock itself is completed and loaded. Such foundation piles are generally cut off and made to act as a unit by a continuous layer or mat of concrete, plain or reinforced, of several feet in thickness around the heads of the piles.

From an economical standpoint, the most favorable use of bearing piles occurs when a practically unyielding stratum can be reached by timber or wood piles of ordinary or market lengths, roughly 20 to 50 ft. long, and the overlying material is compressible, such as soft clay, etc., so as to be readily penetrated by piles, but sufficiently compact to prevent the piles from bending or from displacement laterally. Commercially speaking, the available lengths and kinds of wooden piles are generally limited by the cost of railroad transportation and by the length of railroad cars, although long piles are often shipped in double loads, i.e. with their ends lapping over on another car. Except on our coasts, piling is seldom shipped by boat, as it is a difficult material to stow on account of its length and heaviness as applied to handling through hatch openings.

The line of cut-off on wooden piles in foundations is generally established at the line of permanent moisture, or perhaps a foot above it, for the latter case it being generally believed that capillary action keeps the material in the pile constantly wet as well, and hence impervious to decay. Plain or untreated foundation piles thus cut off at or near the ground water level appear, in numerous examples, to have a practically indefinite life. The life of wooden piles above water, where exposed to air and alternate wetting and drying, is largely affected by the time of year the piles are cut. This feature has not always received the attention it deserves. Scientific experiment shows that it affects not only the durability of the timber, but also its strength. This is also borne out by the observation of practical men who handle and drive piles in large quantities. They state they would depend more on an inferior kind of timber which is cut in

winter when the sap is down, than on the higher grades of timber, not excepting white oak, if cut in summer.

Also, while there are many specifications as to the kinds of wood permissible for use as piles, the well established rule is that any kind of pile which successfully stands modern pile-driving methods with heavy machinery, will support the safe load which it is designed to receive in an ordinary pile foundation. The quality of a pile can usually be judged by the behavior of its head under moderate driving. As driving progresses, the condition of the head also gives some indication of the action of the pile below the surface.

The first step in the design of the foundation for any structure is to ascertain the condition of the soil. As a general rule, the making of borings is valuable, but if the engineer depends upon others to obtain this information for him, he is soon confronted with a great deal of conflicting evidence. In the absence of definite knowledge by previous pile-driving experience at the site, or adjacent to it, borings certainly should be made or test piles driven.[1] The driving of test piles, while generally more expensive, is the more advantageous of the two, as while it does not give samples of the material—that is, the character of the soil—it does give good information on which to decide upon the capacity of the soil.

It is evident, for the purpose of design and economical construction, that the length of piles to go in a pile foundation should be determined in advance. This is where the driving of test piles is additionally valuable. Wood piles cannot be designed, but must be taken as they grow.

If other methods are used to determine the supporting power of the earth and it is desired to compute the size of the pile, it should be remembered that "with the usual methods in effect, in which large initial stresses are to be expected, it is not safe to use piles of diameters which would be just large enough to support the developed supporting power of the earth, nor would it be practicable to secure or drive them." In ordinary pile-driving operations, design is not so absolute as in steel structures, for instance, and some little elasticity is advisable, particularly after the driving of the foundation itself is started and observations are being made. In systematic pile-driving operations over large areas, soft and hard spots in the ground often evidence

[1] A formula is sometimes used which expresses the relation between the safe bearing capacity of a pile and the variable factors which can be observed during its driving (see Appendix *B*).—*Editors in Chief.*

themselves in a manner that is not always developed by the test piles driven previously at regular, but scattered intervals. For example, in case of doubt, on an ordinary pile-driving job, when a little easier driving is encountered, it is better to drive a few additional piles such as are immediately available, rather than to wait for an exact number of larger piles to take the same individual loading.

51. Kinds of Wood Commercially Available for Foundation Piling.—The kinds of wood commercially available for foundation piling varies in different portions of the country. In the Middle West, cypress, tamarack, and mixed hard wood may be obtained in lengths up to 50 and even to 60 ft. In yellow pine, piles may be obtained generally in rather limited quantities up to 90 ft. long. Oregon fir piles, which come in long lengths are seldom used in the Middle West as the freight rate for the long haul is prohibitive.

In the East, yellow pine is easily obtainable in the longer lengths from the Southern states, and for shorter piling various woods from the middle Southern states are often employed.

The hardest piles are white oak and some collateral varieties.

Mixed hardwood piles stand second in the list, and these comprise oak, gum, elm, maple, beech, birch, hickory, pecan, ash, sycamore, locust and chestnut.

The semi-hard woods comprise cypress, tamarack and long-leaf yellow pine.

The soft woods include cottonwood, willow, poplar, cucumber, basswood, hemlock and white pine. These last two varieties are practically extinct in the United States.

Yellow pine and cypress, in addition to their availability in the market, are most suitable for pile foundations, as they are straight, well bodied, and free from large branches, and can be obtained in long lengths.

Oak piles are hard and tough, and stand driving well, but are not so straight and smooth and generally not so well bodied, as they usually do not hold their diameters in proportion to the lengths as pine piles do, being generally over large at the bole or butt end as compared to pine piles and a tendency to run under size at the top; they have the added disadvantage of sinking in water unless rafted to lighter piles or timbers. They are chiefly used for dock work, trestles, etc., as in addition to durability against weather they stand boring and holding by anchor rods,

drift bolts and screw-bolting better than the softer woods allowable for pile foundations.

52. Sizes of Piles.—While large, full-bodied piles are more and more difficult to obtain, due to the exhausting of our forest products, minimum specifications naturally receive most attention. As to maximum specifications, the diameter which piles may not exceed is generally given at 20 in. due to the clearance between the leads of the pile driver being about 22 in. For the foundation of the large grain elevator on the Calumet River at South Chicago, the Chicago & North Western R. R. furnished such large piles, practically all saw logs, that only about half of the 5 or 6 pile-drivers employed on the work could drive piles continually day after day, as it was hard to keep up steam on the boilers to develop enough power to meet the increased friction. In such cases it also costs more per lineal foot to drive piles.

53. Removing Bark from Piles.—The question of removing bark from piles or peeling them, has been much discussed. For dock work, trestles, etc., where the pile is exposed above water, peeling seems most necessary to avoid decay. For foundations, however, there is an increasing tendency to drive piles with the bark on. The growing scarcity of pile material is to a large extent responsible for this. If piles are to be peeled, it should be done in the woods to insure good work, and not at the site of the driving where an expensive crew is kept waiting. An experienced contractor has stated his opinion on the question of peeling piles for foundations as follows: "If the bark is loose, it comes off in driving, and if it stays on, it is as good as the pile itself and helps develop more friction due to its roughness." An extreme case was cited of pulling piles a few years ago from the foundations of the burned Iowa Elevator on the Chicago River, and in which 800 to 900 hardwood foundation piles that had been in place for about 30 yr. were pulled out clean, leaving the bark in the ground. This was mentioned to indicate the intense friction developed between the bark and the ground.

54. Pile-driving Procedure.—Modern pile-driving, as exemplified in the United States, is done mostly by steam power, although gasoline, kerosene, and electric engines are used to a small extent. Horse power was used considerably 25 yr. ago, and is sometimes used still in out-of-the-way places and with a drop hammer. In Japan, Russia, and similar countries where rough labor is cheap

and plentiful, treadmills and even hand hoists are employed to raise the hammer.

The operation of pile-driving may be defined as forcing a pile into a definite position in the ground without previous excavation.

The operation may be most readily illustrated by the case in which a timber pile is driven vertically into the ground by a drop hammer. When piles have been delivered to the site within reach of one of the hoisting lines of the pile-driver, this pile line is made fast to a pile at its head and first dragged, if necessary, near the front of the pile-driver, and then hoisted until it is suspended in air. It is then placed and held between the high parallel members of the pile-driver, known as the leads or guides, in which the hammer slides and is guided in its movements. Next the pile is lowered till its tip rests on the ground, and the pile hammer, which had been hoisted out of the way, is now lowered gently to rest on the head of the pile. Generally in soft ground the pile runs a little under the weight of the hammer. The pile line is then released and actual driving commences; that is, the hammer is raised and released, and at each fall strikes the head of the pile, continuing until the required penetration is reached.

It is evident that some of the work done by the falling hammer is consumed in friction, in crushing or brooming and hitting the head of the pile, and in compressing the pile and the hammer itself, while the remainder causes the penetration of the pile.

55. Pile-drivers.—A pile-driver is an outfit or apparatus for driving piles. It is characterized by the leads—sometimes called leaders—which are upright parallel members supporting the pulleys or sheaves used to hoist the hammer and piles, and to guide the movements of the hammer. Leaders may be of steel or wood. In the latter case the inside surface is generally lined with iron channels to reduce friction and wear. Steel leaders are not used as much as wooden leaders, although they answer satisfactorily for special work, such as cast-in-place concrete piles. In the more usual type of work the dragging and hoisting of a heavy pile over rough ground sometimes causes the pile to strike the leaders violently, which will deform steel leads more quickly than the more springy wooden ones. The steel leaders are also more apt to shake loose at the joints from hard driving than the wooden frames and are not so much favored for general utility purposes.

On floating pile-drivers the top of the leads are arranged so that pulling blocks or falls may be set in place for the purpose of pulling round timber piles and wood or sheet piles, as in coffer-dams for bridge foundations or for removing old docks, etc. The buoyancy of the hull of the pile-driver is thus employed to pull out piles.

The pile-driver leads are braced in position by back stays and by horizontal and diagonal members to form the tower, the

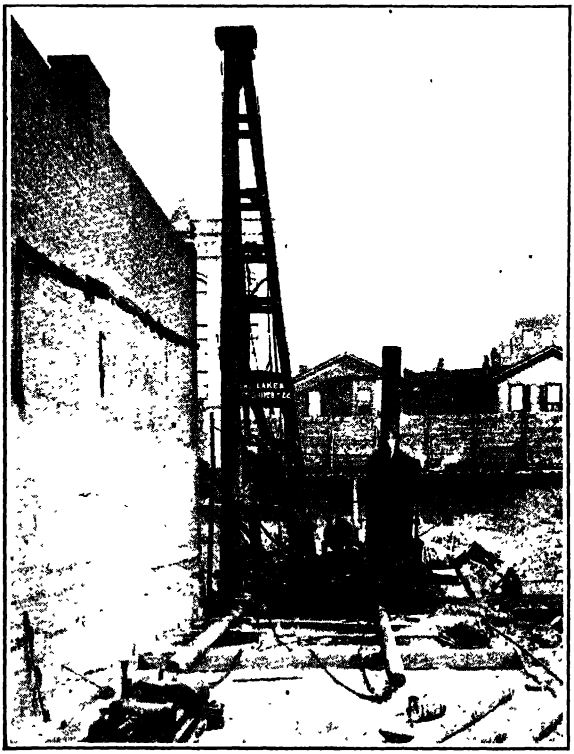


Fig. 21.—Pile-driver mounted on rollers.

whole resting on a bed frame of horizontal sills which generally extend far enough back to carry the hoisting engine and boiler. This whole outfit is then either mounted direct on rollers or on a turntable which is set on rollers. The first type of driver is illustrated in Fig. 21. Fig. 22 gives a close-up view of this general type of driver. The driver is moved sidewise on rollers by

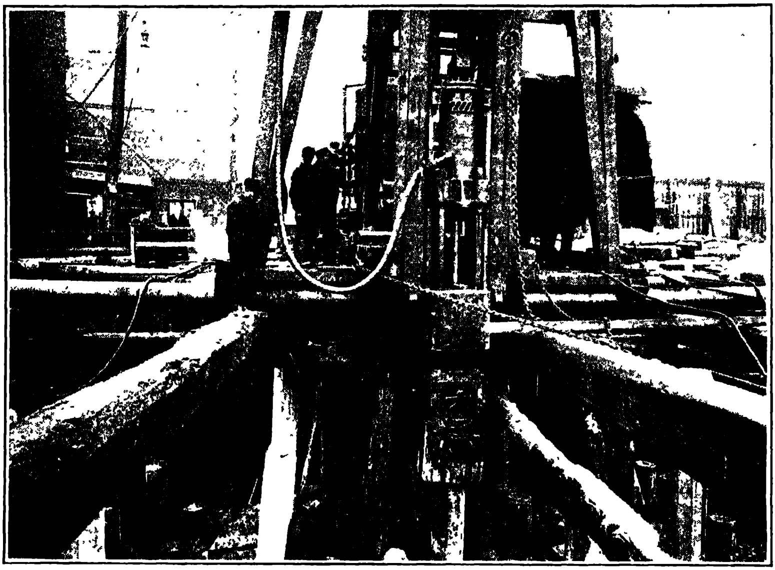


Fig. 22.—Close-up view of the general type of pile-driver shown in Fig. 21.

Fig. 23.—So-called Chicago type of heavy swinging pile-driver.

winding with the engine on the chains shown at the ends of rollers. To move forward or back the chain is run to some intervening object in the direction of proposed travel and beyond it, so that when the engine winds the chain, it slues the end roller around as desired, and this operation is then repeated for the other end of roller.

The second type of driver is called a swinging pile-driver. Fig. 23 shows the so-called Chicago type of heavy swinging driver, which is transported and used on a scow, but is used principally as a land or shore driver. In Fig. 23 it is shown driving long wooden foundation piles for the back abutment of Michigan Avenue Bridge, Chicago. It has leaders 75 ft. high on a steel base called shoes. In driving a foundation on land these shoes carry the driver on blocking, which holds the lower end of the leaders about 4 ft. above the ground level. About 15 ft. of the leaders is taken up by the large steam hammer employed. This allows piles up to 60 ft. length to be driven on land, and upwards of 80 ft. length in water, depending on the depth. This driver swings on a circle of 40 ft.—that is, it can reach out and drive a pile 20 ft. to one side and then swing 180 deg. and drive a pile 20 ft. to the other side, or it can drive a pile 20 ft. ahead of its swinging center and swing completely backwards and drive another pile 40 ft. back from the last driven pile. It carries a Warrington steam hammer weighing 5 tons.

Fig. 23 shows the heavy iron straps at the foot of the leaders and the back stays, which allow the driver to be dismantled for transporting the leaders as a unit on a truck or by freight cars into a building foundation. While outfits of this kind appear very heavy, long experience proves they accomplish the hard work and large output required of them in a highly satisfactory manner. The particular pile-driver shown has driven over 100,000 piles, aggregating about 5 million linear feet of piling, and has had but one new set of leaders since it was built As the tendency is towards concrete piling in foundations, the heavy driver is more and more necessary to handle and drive these heavy piles. Practical pile-driver men maintain that a heavy outfit best controls the direction of the pile that is being driven; in other words, that the driver forces the pile instead of the pile forcing the driving machine. The shoes on this driver are 12 ft. wide, which allows it to be taken in and out alleys and in narrower spaces than drivers with more spread of base. The Milwaukee

type of driver, for instance, is 20 ft. wide and cannot be gotten in and out of many foundation sites as handily, if at all, as a narrower driver base. These wider drivers also require twice as much running timber on which to move them, and they are also harder on rollers.

The type of driver shown is considered the best utility driver for general contract work, as not alone building foundations, but

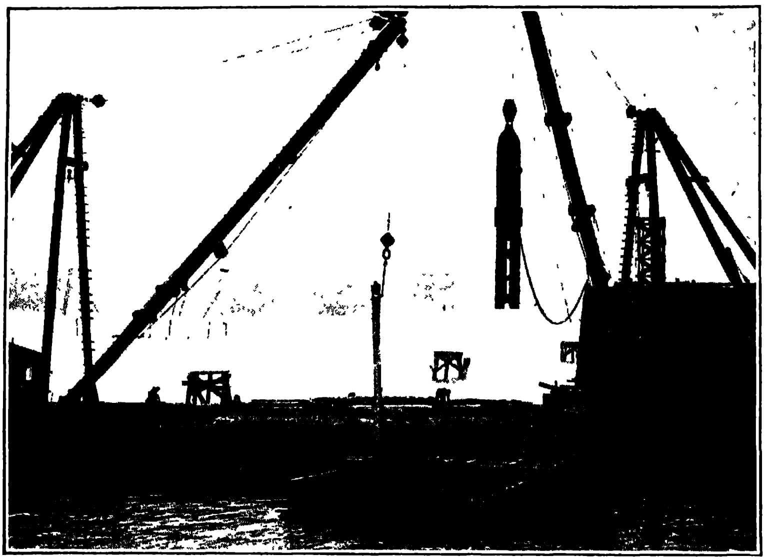


FIG. 24.—Floating derrick with hanging leads driving bridge foundation piles in a cofferdam.

cofferdams, docks and sheeting of all kinds can be driven with this outfit ashore or afloat. Its only limitation is that it cannot pull piles, this requiring a floating driver with the leads attached to the hull.

Hanging leads are often used to drive piles with ordinary derricks or locomotive cranes. Fig. 24 shows a floating derrick with hanging leads driving bridge foundation piles in a cofferdam by means of a long steel guide pipe reaching to the river bottom, and with a wooden follower inside of this pipe. This device was employed for a series of bridge piers for the Pennsylvania R. R. Company's new bridge across the Maumee River at Toledo, Ohio. While hanging leads have their use in driving piles in deep trenches or through contracted openings, or in the

bottoms of cofferdams containing a large amount of internal bracing, their general use is limited, and they are looked on mostly as a make-shift.

Track pile-drivers for railroad service have been developed to a high degree of efficiency. They are used largely on trestle work

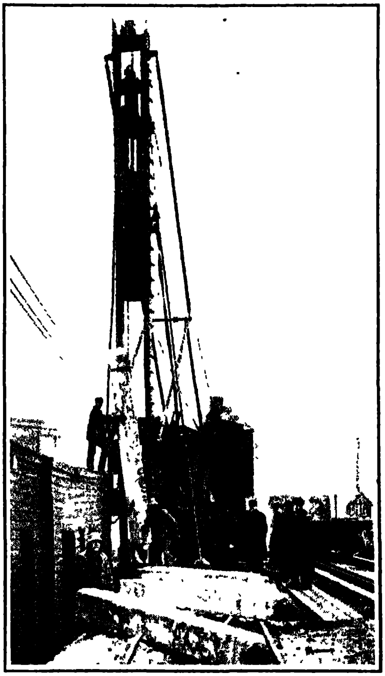


Fig. 25.—Track pile-driver for railroad service.

and track elevation projects for railroads, and for bridge and culvert foundations, but are not used to any great extent on foundations for other structures (see Fig. 25).

56. Drop Hammers.—The drop hammer is that type of pile hammer which is raised by a rope or steel cable and then allowed to drop. Its essential features consist of a solid casting with jaws on each side which fit into the guides of the pile-driver leads; it has a pin near the top for attaching the hoisting rope or nippers, as the case may be. It has a broad base on which it strikes the pile, the idea being to keep the center of gravity of the hammer as low as possible.

Fig. 26 shows a drop hammer of modern design employed in a dry-dock foundation, also a pile cap, which will be referred to later, it being shown below the hammer proper. The hammer is made as long as practicable to increase the bearing in the leads, and all corners are rounded. The jaws are given as little play as

Fig. 26—Drop hammer of modern design.

practicable to reduce the jar on the driver when a blow is delivered to the pile. The bottom of the base of the hammer is made slightly concave when the hammer is to be used to strike the head of the pile directly. When a pile-cap is to be used, the base of hammer is made flat.

The weight of drop hammers used in the United States to drive wooden piles varies from 2000 to 4000 lb., the latter named weight of hammer being about 7 ft. high. For very short piles a weight as low as 500 lb. is sometimes used. The weight of hammer to be used depends upon the length and size of the piles and the character of the ground to be driven through. Most contractors have a number of sizes to select from for the varying

conditions of their work. Hollow hammers are sometimes used, so that they may be loaded within a limitable range to give the varying weights. Pig iron or junk is used for loading. These hollow hammers are also of cast-iron like the solid hammers already discussed, the casting being about 3 in. thick. The chief use of these hammers is not in pile foundations, however, but for driving sheeting, and they are generally provided for this purpose with a special lip or projection of about 3 ft. which allows the hammer to be worked in the usual leaders of the pile-

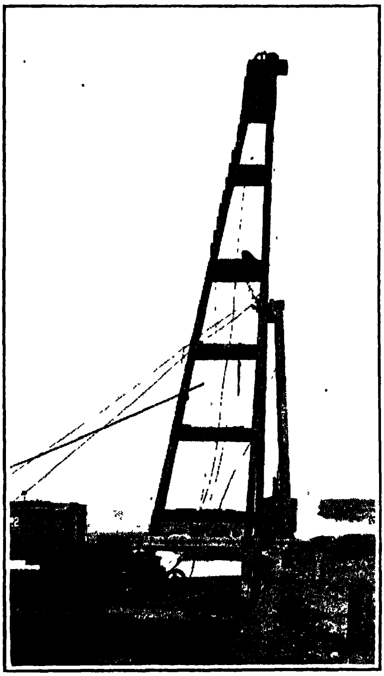


FIG. 27.—Showing false leaders for driving sheet piling.

driver and at the same time be able to get vertically over its work without the use of false leads. The false leaders for driving sheeting are shown in Fig. 27 and are not as hard on the permanent leaders as the overhanging hammer, but it takes considerable time to put on and take off false leaders as compared with the overhang hammer, which can be put in the regular leaders in a few minutes. These hollow hammers which are used to drive triple-lap or so-called Wakefield sheeting in heavy dock work,

cofferdams, etc., weigh about 3600 lb. light and about 4500 lb. fully loaded. It is well to note in this connection that as a rule wooden sheeting is more generally driven with a drop hammer, as better control of the blow is claimed.

57. Steam Hammers.—The steam hammer for driving piles is one which is automatically raised and then dropped a short distance by means of a steam cylinder and piston held in a frame in

FIG. 28.—Extra large Warrington hammer.

the leaders of the pile-driver so that it follows the pile down in driving. James Nasmyth of England invented the first steam hammer about 1850. In the United States the Vulcan Iron Works of Chicago has been building for about 45 yr. with continuous improvements, the steam hammer now known as the Warrington hammer. An extra large Warrington hammer is shown in Fig. 28. It weighs 16,000 lb., the striking part weighing 7500 lb. It is employed for driving large concrete piles, weighing 5 to 10 tons. It has a special base to receive a rope mat to prevent spalling off at the concrete in the head of the pile during hard driving.

The steam hammers are of two general classes—single-acting and double-acting. The single-acting is the older and better known type. Steam pressure conveyed from the boiler of the pile-driver by a steam hose is used to raise the striking part of the hammer, which then falls by gravity. The force of the blow depends upon the length of the stroke and the weight of the movable part. The number of blows per minute depends upon the steam pressure, varying from 50 to 60 per minute.

In the second class of hammers—the double-acting—steam pressure raises the hammer and also assists the action of gravity on the down stroke. The force of the blow and its rapidity depend upon the steam pressure. The double-acting hammer is lighter, more compact, takes up less room in the leads, and operates with more rapidity than single-acting steam hammers of the Warrington and Cram type. Examples of double-acting steam hammers are the Arnott, McKiernan-Terry, Goubert, Industrial Works and New Monarch. Compressed air may be used to operate the Arnott and McKiernan-Terry types. The weight of steam hammers runs from about 200 to 16,000 lb. The smaller sizes are used mostly for driving light piling, and have no place in foundation piling work where the largest hammer practicable is generally the most efficient. The double-acting hammer in general delivers 100 to 120 blows per minute, or about double the number of the single acting hammer.

In driving wooden piles with a steam hammer, it rests with its frame upon the pile, the pile head being trimmed to fit into the recess or base of the frame, the frame having on its sides angles or channels which engage the leads in the driver. In turn, the frame guides the hammer movement. In some makes of hammer the frame entirely encases the striking part. The dead weight of the frame generally is many times the weight of the striking part, and also much heavier compared to a drop hammer used under the same circumstances, and hence helps to keep the pile in motion after it is started by the blow. As the blows follow rapidly, the pile is kept in continual motion, which is an important aid. The small vibration set up in the pile also assists its penetration.

58. Drop and Steam Hammers Compared.—The advantages of the steam hammer over the drop hammer are as follows:

1. The pile is kept in position more firmly and guided better in driving.
2. Damaging the pile by brooming and splitting is more apt to be avoided, as piles must be headed more often with a drop hammer due to this

cause. The use of softer woods for piling is thus allowed with a steam hammer.

3. Driving is equally effective for any position of the piling in the leads.
4. A pile may be driven several feet (with some hammers 7 or 8 ft.) below the bottom of the fixed leads without the use of extension leads or a follower. This often saves several feet of pile length which is important when material is expensive, and in the longer lengths of piles the owner pays not alone for the extra length, but an additional price for every foot of the pile as compared to a shorter pile. For instance, if the material in a 40-ft. pile costs 20 cts. per ft., the 45 ft. will generally cost about 23 cts. per ft., meaning a saving of $2.35 in material per pile if a 40-ft. pile can be followed down, instead of driving a 45-ft. pile and wasting some of it. This also saves the expense of cutting off piles.
5. The rapidity of action keeps the pile in motion in all but the hardest driving, and as in foundation work almost every foot of the pile must be driven, compared to driving a pile in water, it means faster work. In a foundation three piles can usually be driven with a steam hammer while one is being driven with a drop hammer, as much time is lost in hoisting the drop hammer between blows.
6. It can be used in places and under conditions where a drop hammer cannot be used successfully.
7. Less injury is caused to adjacent foundations and less breaking of glass and cracking of plaster in nearby buildings.

In Chicago, which is a pile-driving locality, steam hammers are used almost exclusively for foundation work and for bearing piles and anchor piles for docks. In the language of pile-driver men it coaxes the piles down better than the drop hammer.

In lake work, however, or generally in water work, the drop hammer is preferred for several reasons. Among these are the following:

The drop hammer is best on easy driving, and this is usually the case in water work, as the distance driven into the ground is less than in foundations. Also, on water work the tops of the piles are generally left at 4 to 5 ft. above the water surface, and hence the advantage of a steam hammer which can drive below the leads is not necessary. In fact, the steam hammer is so heavy, being generally about 5 tons weight as against 2 tons for a drop hammer, that its position of leverage in the leads when on a hull, puts the ordinary pile-driver scow down by the head, so that it is impossible to leave piles standing 5 to 6 ft. out of water. Also, in varied work, where it is often necessary to take the steam hammer in and out of the leads and substitute a drop hammer, or vice versa, the weight of the steam hammer (generally about 5 tons for

driving wood piles) is required to be handled and a safe and strong place must be provided to receive the hammer. For these reasons, changing from a steam hammer to a drop hammer generally consumes 1 to 1½ hr. of time of a crew, and is, therefore, expensive and hence objectionable.

Another objection to the steam hammer in such work as pile-piers, etc., is that they generally consist of close-driven lines of piles on which the sides of the steam hammer will catch unless the work is done by backing away instead of working the driver sideways, as is the usual practice. In driving foundation piles, the spacing of piles, which is never closer than 2½ to 3 ft., does not render this feature of the steam hammer objectionable.

59. Reducing Tendency to Lateral Movement of Pile.—The butt of a pile should always be cut off square, so that the impact of the hammer may be distributed uniformly over the surface. Since the butt end tends to change its position slightly in the leads during driving, it has been found advantageous to make the lower surface of the drop hammer slightly concave. This provision counteracts the tendency toward lateral movement of the pile to some degree.

60. Protecting Pile Head.—To prevent splitting and reduce brooming, the head of a pile may be hooped with a pile ring, to receive which the pile is neatly chamfered down so that the first blow of the pile hammer puts the ring in place. The rings range in size from 2 × ⅜ in. to 4 × 1 in. The diameters, of course, vary to suit the different sizes of piles. They are made of the best quality of wrought iron and generally last to drive about 100 oak piles and about twice that many soft wood piles. To remove the ring, a cant hook is used alone, or with the pile line attached to apply steam power. When a pile brooms too much in spite of the ring, the remedy is to saw off the broomed portion to a solid surface and replace the ring.

A more modern and more effective and less expensive method of protecting a pile head is the use of a device known as Casgrain's pile cap (see Fig. 26). The chamfered head of the pile fits into a tapered recess in the bottom of a separate casting below the drop hammer, and a short cushion block of wood, preferably oak, is fitted into an upper recess. The cap has jaws on its sides to engage the pile driver leads like the hammer itself, and so holds the head of the pile in position and guides it in driving. After the

pile is driven, the cap is hooked to the hammer casting and is raised with the hammer.

In pile pier and pile breakwater work in the Great Lakes district, neither a ring nor the more modern cap or bonnet is used with a drop hammer, as the use of the bonnet consumes a lot of time. When a pile is dropped to the bottom to begin driving, it generally is easily controlled by the two loader men using toggles or car stakes to pry the pile into line for driving. Another disadvantage is that the Casgrain caps or similar styles of bonnet have a tendency to jump off a pile and are easily lost in lake work. On river work where dock lines are to be driven, a bonnet is oftentimes used to control the driving of a pile, and in such cases, in addition to its usual hooking arrangement, a line is put around it, passing over the top of the hammer, so that if the bonnet or cap jumps off into the water it can be reclaimed. A bonnet is similarly employed when piles are driven on a slope and have a tendency to drive away from the hammer.

Years ago, a flat steel plate was spiked on the pile head to receive the blow, and a later device was a dished or cup striking plate for the same purpose. A still better arrangement is used with some makes of steam hammers, for example the well known Warrington steam hammer is often provided with a so-called McDermid patent base (shown in Fig. 23) in which a recess is provided for a forged plate about 2 in. thick, called a beating plate, which is inserted through a slot in the side, covered by a slide to hold it in place, thus avoiding danger to the crew which occurs with a separate plate. As the slot that takes the beating plate is 4 in. high, and as the plate is pushed to the top of the slot in driving, the top of the pile sometimes brooms up, particularly in hard driving, and fills the lower part of the slot with wood fiber and makes it difficult to get the base off the pile.

61. Followers.—If a pile is to be driven below the leads or below the ground or water surface, a follower is usually employed. Briefly, it is a member interposed between the hammer and the pile to transmit blows to the pile when the latter is out of the leads. One of the simplest forms is a short pile or dollie of white oak, with a projecting band of iron on its lower end to keep it on the pile head (see Fig. 29). A cylindrical casting, with a horizontal division or web in the middle is often employed, one portion of the casting being bolted to the follower, or sometimes having an extra strong pipe cast in it to avoid the use of bolts, while the other end

fits over the pile head. The upper end of the follower is held in position by the recessed base of the steam hammer, or by a pile cap if a drop hammer is being used. A follower absorbs a considerable portion of the energy of the hammer,—sometimes as

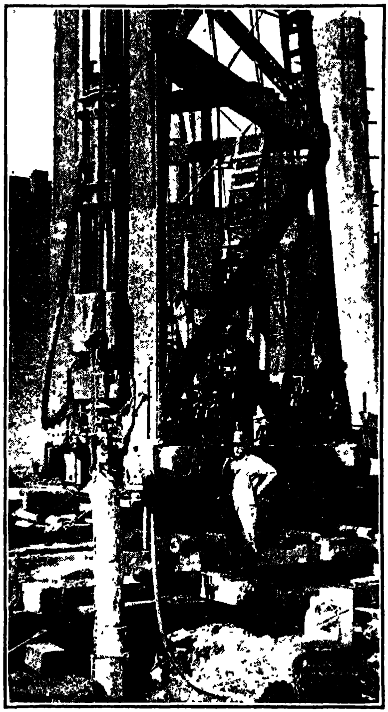


FIG. 29.—Showing use of short pile follower.

much as 50 per cent. As a follower is apt to stick in the ground, particularly in clayey soils, the practical limit of following is about 5 ft. Some years ago a device employing steam at boiler pressure to the foot of the follower was used with success in the Chicago district to break up the suction of the ground around the pile head, but this device seems to be no longer used.

The driving of piles for the bridge piers in Maumee River at Toledo, by means of a long wooden follower working inside of a large iron pipe used as a guide (Fig. 24) is a rather unusual illustration of the use of a simple follower.

62. Blunt vs. Pointed Piles.—The foot of a timber pile should be cut off perpendicular to its axis, as it facilitates driving it true

to line or position. In soft ground where driving is easy, it is hardly necessary to sharpen or point the pile. If it penetrates soft material and rests upon hard stratum so as to act as a column, the blunt end has the additional advantage of providing larger bearing area. Great care should be used in the latter case to prevent over-driving, which will shatter or crush the feet of piles and seriously impair their supporting power.

In driving a pile with a blunt end, a cone of compressed earth forms under it and acts to a large extent as if the foot were pointed. Some experienced pile driving men claim that even in driving through hard material, a blunt pile will keep more nearly to position than if pointed. In hard material or in driving on a slope, pointing is necessary, and it should be sharpened to the form of a truncated pyramid, the end being 4 to 6 in. square. The length of the point should be about twice the diameter of the foot. In compact material, the bearing power is practically the same with a point as without. In driving through debris or old grillage, etc., it is well to point piles for successful driving or even to use metal shoes.

63. Pile Shoes.—Many engineers and contractors condemn the use of pile shoes on the basis that they are not needed in soft driving, and that in difficult material the shoe strips off from the unevenness of the point of application in hard driving. Pile shoes are difficult to properly fit to a pile and probably are condemned mostly as a loss of time.

64. Driving Piles with Butt Downward.—While it is the general practice to drive piles with tip downward, special conditions occasionally make it advisable to drive them with butt downward. In very soft ground, the larger area afforded by the butt of the pile will often be found to carry the load. Another condition occurs when it is difficult to keep a pile down after being struck by the hammer. The pile begins at once to rise, lifting the hammer with it, and may even shoot up into the air when the hammer is raised. A typical instance is in driving range piles in deep water for dumping of stone core for breakwater projects, etc.; in such cases the range piles are so affected by the buoyancy of the deep water, as to make it necessary to drive them butt down.

65. Splicing Piles.—It is sometimes necessary to use longer piles than can be obtained in the single sticks. For this purpose, two piles may be spliced together end to end by timber fish plates bolted on four sides of the piles, or a metal sleeve may be used in

the form of a heavy pipe. Half lap joints fastened with bolts generally prove unsatisfactory because of the lack of lateral strength and stiffness. However, in swampy ground, one pile is sometimes driven on top of another with only an iron dowel pin connecting the two.

Pile splices are sometimes used where piles can be driven only in short sections due to limited head room. But in general, this is more apt to be the case with steel sheeting inside of buildings, for pits, wells, shafts, etc. than for foundation piles.

66. Cutting Off Piles.—In water work, cutting off piles is done with a circular saw mounted on a vertical shaft, which may be operated in the leads of a pile-driver by the pile-driver equipment.

In the average land foundation, also in cofferdams, where the piles are cut off after the excavation and pumping is done, the ordinary cross-cut saw, cut into half lengths, tending to make it short enough not to interfere with adjacent piles, and to work in the corners, is found to be more economical and more satisfactory from the standpoint of workmanship even with high-priced labor, than most machine devices.

67. Preservation of Piling.—Chemical preservation of piles usually is not a factor in pile foundations for buildings, where the wooden piles can generally be cut off at ground water level and thereby indefinitely preserved. There is a tendency, however, to use creosoted piles in dock foundations and there is, of course, in general, urgent necessity for creosoting in ocean districts where the teredo is active—for instance, on the Pacific, the South Atlantic, and the Gulf Coasts and tributaries. The higher the water temperature, the more active the teredo. As knots cannot be well creosoted the teredo is likely to enter the pile at such places and damage it. Hence, the tendency lately is toward a concrete armor on untreated wooden piles, this outer covering of concrete extending from just below the ground line to the high water line. Where piles are to be placed entirely by jetting, and hence not jarred by driving, a protection made by cement-gun work on wire mesh reinforcement around a wooden pile is found effective.

Inasmuch as these marine borers are active only in salt water, there seems very little reason for employing creosoted piles in foundations in other districts, considering also the delay and additional expense of creosoting. But the railroad companies seem to favor creosoting of piles for docks and foundations in

various districts through the United States. Although a creosoted round pile behaves the same as an untreated pile in driving, experience shows that pine sheeting for docks when creosoted breaks more easily under the hammer than if untreated. The brittleness appears to be due to the creosoting process. In the Great Lakes district the portion of timber docks above water, that is the round piles, timber and sheet piling, are often treated with several applications of crude oil, which appears to lengthen the life of the timber considerably.

68. Order in Which Piles Should be Driven.—In driving a foundation, particularly when it contains a large number of piles rather closely spaced, driving should always progress toward the line of least resistance; for instance, away from an existing building—not toward it, and toward a river or lake—and not away from it. Instances may be recalled of pile foundation piers near rivers which were driven first and the main foundation landwards of them driven later. In such cases, it was generally found that the river piers were forced outward by the later driving.

Where there is no apparent direction of least resistance, say for instance in driving the foundation of a large gas holder in an inland situation, the inner circle of piles should be driven first and progress made outward over the entire area. A possible exception to this general rule may be the case of such soft ground that the outer piles should, of necessity, be driven first and the inner piles last, in order to develop all the friction possible. In such cases, this will be recognized as a combination of driving and setting piles. In ordinary cases, however, this procedure would soon show its effect by the heaving of the enclosed ground and the possible raising of piles already driven. The heaving of ground between piles is a different proposition, and allowance should be made for this in elastic soil, such as potters clay or water-bearing clay which has no outlet for water. The amount of heaving is, of course, more or less of a guess and depends upon the number of piles driven, their spacing and also the character of the soil as shown by previous experience in similar cases, but it is generally customary to make the general excavation about 1 to 2 ft. deeper than shown on the plans to allow for heaving and to avoid the expense of excavating later the soil compacted by driving, and particularly from digging out between piles, which means hand labor.

69. Lateral Springing of Pile.—In driving piles in some kinds of clay, the lateral springing of the pile under the hammer blows makes a hole slightly larger than the diameter of the pile; in a foundation this allows surface water to find its way to the foot of the pile and thus reduces both skin-friction and the bearing value of the clay under the foot of the pile. At times it causes settlement of piles under heavy loads, particularly moving loads. A very important, and often overlooked point in bridge foundation work is that comparatively small leaks from river water, etc. following down a pile often make trouble in a cofferdam or shaft.

70. Use of Water Jet in Driving Piles.—The use of the water jet in pile-driving operations differs radically in principle from driving with a hammer. Briefly, it consists in displacing the material at the proposed location of the pile by means of one or more water jets.

While the water jet may be used to advantage in any material that will settle around the pile after the jet is withdrawn, the best results are obtained in pure river, lake or ocean sand. The simplest form of single jet under moderate pressure will generally answer in such conditions. It takes but little time to sink the jet and the pile, the sand packs around the pile quickly after it is in place, and very little driving with the hammer is needed except in the case of a clay or harder bottom beneath the sand, when driving by the usual methods is necessary to penetrate these lower strata.

Although the water jet gives good results in driving piles in mixtures of sand, silt and gravel, and also gives fair results in loam, marl and even in some clays and harder materials, contractors continuously using the water jet find that its greatest field is in pure sand. When it is considered that sand offers a high resistance to a pile when driven with a hammer alone, the principle of the water jet is, therefore, highly valuable in this material. With a jet a pile may be sunk in sand without injury, while on account of the long and heavy pounding necessary to get it down, it is difficult to avoid injuring a pile driven into sand without the aid of a jet. The time saved as against the slow process of driving with a hammer in sand is astonishing. The energy saved is also considerable.

In using a water jet the quantity of water is more important than the velocity. The velocity must be enough to excavate the sand and make it alive and quick, while the volume of water must

be sufficient to force the water to escape by rising to the surface and bringing material with it. As one pile-driver operator expressed it, the trick is to make a hole in the sand the length and the diameter of the pile, then pull out the jet and drop the pile into the hole. In pure sand most experienced operators of the jet never drop the pile with the jet. They shove the jet down first, haul it up and then drop the pile in place. If clay underlies the sand, they do not try to jet into the clay stratum. As soon as the nozzle of the jet strikes the clay, it jumps, which is the operator's signal to pull it up, as it is no use to pump farther, as the clay will clog the jet. Straight driving into the clay is then resorted to after the pile is set in the sand.

The single jet equipment for water-jetting successfully in sand generally consists of a straight iron or steel pipe about 2½ in. in diameter, connected by a flexible hose of the same diameter to the discharge end of a force pump, which provides water under pressure of about 100 lb. The pump is operated by steam generated from the boiler of the pile-driver or in some cases by a separate steam supply.

To increase the velocity of the water and thus loosen the sand or earth, the lower or free end of the pipe is generally drawn down to form a nozzle. Nozzling down to 1½ to 1¾ in. diameter is found to get the best results. One of the great mistakes in the use of a water jet is to employ too small a pipe or to nozzle it down too much. Often the using of too small a pump spoils the results, as quantity of water is essential. Inadequate equipment is probably one of the main reasons why the water jet process has not come into more extensive use in pile-driving practice. In driving through gravel greater water pressures are needed than mentioned for sand. In such cases the water jet often washes out any sand and small gravel, and leaves the larger material—often cobblestones—to settle in the hole and interfere with the driving of the pile. Sometimes this can be remedied by increasing the volume and pressure of the water. The movement of water in jetting appears to be confined to a small radius horizontally—much less than the usual 2½- to 3-ft. spacing allowed by good practice for the driving of wooden piles in foundations; this is, of course, important as not affecting adjacent piles during construction.

For jetting concrete piles the jet pipe is often cast in the center of the pile to deliver water to the foot of the pile in order to displace material there; also, side jets running from the center

edges are often cast in the pile to further assist in floating the material and in reducing surface friction.

The use of a water jet is especially valuable in driving concrete piles, both to save the energy and time required to drive long and heavy piles, but also to avoid possible injury to the pile by the use of the hammer. It is evident the pre-molded type of concrete pile is ideal for conditions of sand, quicksand, light gravel, etc.

71. Number of Piles That Can be Driven in a Day.—The number of timber piles which can be driven in a day by one pile-driver crew depends upon many factors. The result is affected by the size of the piles, the distance apart the piles are driven, the depth to which the piles are driven, the kind of ground encountered, whether soft driving or hard, the kind of hammer used—whether drop hammer or steam hammer, the weight of hammer relatively to the weight of pile, whether a water jet is employed or not, the training and experience of the crew, the kind and condition of the pile-driving equipment, the experience or non-experience of the inspectors directing the pile-driving for the owners.

The spacing of the piles is not usually given the attention it deserves in considering pile-driving output, as moving of the pile-driver from one position to another where the piles are spaced far apart, consumes a large part of the working time. This and the snaking of piles to the leader, setting the driver in exact position, placing the pile in the leads, etc. also take more time than is usually realized unless time studies are made.

As to maximum performance per day in pile-driving, many records can be cited of 100, and even in exceptional cases as high as 200 piles driven by one outfit and crew in a 10-hr. day. During the recent war, reports were published of as high as 220 piles 65 ft. long being driven in one shift of 9 hr. at one of the new shipyards being constructed on the Atlantic Coast. This was probably in extremely soft ground.

The maximum performance is, however, grossly misleading as compared to the average number of piles driven per day over a long period. A large contracting organization that drives thousands of piles every year and operates over a large territory with varying conditions of ground formation gives 30 to 35 foundation piles as their average output per day of 8 hr. This is under average conditions from clay driving to jetting and driving. Many days 40 to 60 piles are driven, but the one or two days in the month that only 20 piles, or perhaps less, are driven, cuts

down the average to the 30 or 35 piles stated, and which they consider is good progress and can be exceeded only under exceptional conditions. Within ordinary limits the length of piles does not greatly affect the number of piles that can be driven in a day. Ordinarily speaking, about as many 40-ft. piles can be driven in a day as 35-ft. piles; in other words, the driving of the additional length is not the limiting cause, but rather the moving around and the setting of driver in exact position for driving, the getting of piles to the driver, and other delays, exclusive of actual driving.

72. Cost of Pile Driving.—On account of the varying features that have been outlined previously and the great number of elements entering into progress in pile-driving, it is difficult to give costs that are of real value in estimating work. Statements of the cost of driving piles are given in the engineering periodicals and in handbooks of cost data, but are apt to be misleading unless the local conditions, the methods, pile-driving equipment and other factors are given in detail. Estimates on foundation piling are generally stated in a price per lin. ft. of pile driven, the length of pile being considered as that measured in the leads just before driving.

A rough estimate of cost of foundation piles is sometimes made by doubling the cost of the plain material in a wooden pile (untreated). For instance, if a 40-ft. mixed wood pile can be bought delivered on cars at site at the rate of 20 cts. a lin. ft., the price of 40 cts. a lin. ft. for the pile in place would be the commercial price approximately.

73. Driving Foundation Piles for Bridge Piers.—In driving foundation piles for bridge piers inside of cofferdams this operation is often done from the top of the cofferdam but is not considered good practice by the best constructors, if it can possibly be avoided as it is apt to cause damage or even collapse to the interior bracing of the dam. It is considered much better practice to drive the foundation piles before the dam is closed in. Often the back line of the dam, consisting of piles and sheeting, is driven and also the sides of the dam are brought out partly or the entire distance and then the foundation piles are driven with a floating driver, if possible, but otherwise by a land driver and then the front or remaining sides of the cofferdam are driven and the bracing placed. The additional cost of excavating around the driven piles under this method is generally a saving in the end as

compared to the damage that may be done to a large and deep cofferdam by driving from a machine placed on top of the bracing or by hanging leads.

74. Use of Batter Piles in Foundations.—Batter piles, sometimes called spur piles, and in Europe called slanting piles, are often driven in the foundations of abutments for arch bridges to resist the horizontal components of the reaction. For simple truss or girder spans a few batter piles at each side of the pier are sometimes used when the weight of the pier itself does not provide sufficiently for effective traction. Dock walls are a more common illustration of the employment of batter piles. They are used in addition to the tie rods with anchor piles, and sometimes, although infrequently, are depended on entirely in place of anchorage.

Accidents often occur by failure to provide batter piles to relieve vertical piles from such stresses. Vertical piles in permanent structures should be protected against the action of lateral force by the use of batter piles or other devices. The following is an illustration.

The large power house on the bank of St. Mary's River at Sault Ste. Marie, Mich. was supported by vertical foundation piles of timber, which later showed lateral deflection, the whole structure moving toward the river due to a 20-ft. difference of water-head; the Lake Superior water level being brought to the land side of the Power House by means of a canal, and the water when released at the river side being at the lower level or that of St. Mary's River. As it was necessary to keep the power house in operation during reinforcing of the pile foundation, it was decided to drive a number of inclined shafts or struts to take up the heavy horizontal load. Tubular steel piles were jacked by sections into the river bottom and filled with concrete to form large batter piles.

CONCRETE PILES

By Walter Cahill

The phenomenal growth in the American Portland cement industry, commencing about the year 1900 and making cement increasingly cheaper, is largely responsible for the introduction of concrete piles. The increasing cost of timber piles, due to the decrease in our timber resources, as well as the short life of such

piles when their upper portions are alternately wet and dry, as in docks and trestles, and in some instances foundations of bridges and other structures, have given concrete piles of various types an opportunity for demonstration.

Concrete piles are found to arrange themselves in two broad classes: (1) Those which are molded or cast to a regular form, and which, after curing, are handled and driven similarly to wooden piles. These are usually called pre-molded or pre-cast piles. (2) Those cast or formed in place, either with or without the use of casings. These are generally called cast-in-place piles. The casings are usually of thin metal, which remain in the ground until destroyed by corrosion.

Pre-molded piles were first introduced in Europe by Hennebique towards the close of the nineteenth century. The cast-in-place pile was invented in America by A. A. Raymond, being first used in a building foundation in Chicago about 20 yr. ago, and this type came into considerable use before the many advantages of pre-molded piles were generally recognized.

Cast-in-place piles are not generally reinforced with steel bars or rods, although this feature can be added. Pre-molded piles, however, are generally a reinforced pile, using longitudinal steel rods, or rods in combination with lateral reinforcement of wire hoops or spiral wrapping. In cross-section they are square, circular, hexagonal or octagonal. Piles of triangular section have been used in Germany. Those with a circular section generally have no taper; the other sections are usually tapered their entire length. The tendency in general is to use a decidedly smaller taper in pre-molded piles than in cast-in-place piles. The railroads have mostly adopted the untapered pile as a standard in pre-molded piles.

A combination of a timber and a concrete pile also is sometimes used.

75. Advantages of Concrete Piles as Compared with Timber Piles.—As timber piles must be kept constantly submerged to remain preserved from the decay which results from alternate wetting and drying, they are generally cut off below the permanent ground water level in the usual run of foundation work. This often involves the cost of additional or deeper excavation. When the water level is lowered by changes in drainage, lowering of sewers or construction of subways in large cities, the exposed portion of wooden piles becomes liable to rot, and by

weakening, may possibly cause settlement of foundations, involving expensive changes to correct.

The most important advantage of concrete piles is that they are equally durable in dry or wet soil, their durability being independent of ground water level and the rise and fall of the tides. Also they can be driven through rotted vegetation without the misgivings as to timber piles so employed. A concrete pile is not subjected to the ravages of the teredo worm; in salt water, infested by the limnoria or teredo, timber piles require chemical or mechanical protection, either of which is expensive. They also have a considerable advantage over timber piles due to their larger size, thus enabling a material reduction to be made in the number of piles required to support a given structure. Engineers consider the approximate loading of timber piles to range from 10 to 20 tons each, while concrete piles may be loaded from 20 to 50 tons each. However, a concrete pile is more expensive in first cost and generally is more troublesome and expensive to handle and drive, on account of its greater weight and relatively less flexural strength. They cannot be driven as rapidly as timber piles, but the number required may be enough smaller to save time as well as cost. Their use, being independent of ground water level, avoids extra excavation and, as a direct consequence, effects a saving in masonry walls and footings. This is often the largest factor of saving, particularly if the tops of the concrete piles can be placed more than 3 ft. higher than for timber piles. Less excavation and smaller footings also save time in construction. Also, they sometimes have a secondary influence on the cost of the foundation by reducing the weight to be supported. The reduction in excavation, especially in depth, may lessen the amount of shoring, sheeting, pumping and backfilling, which items are often difficult to estimate, and, therefore, contractors, as well as engineers, prefer to eliminate them as far as possible. All of these advantages can be considered adequately only in the detailed design and estimate of cost for a given structure, but numerous examples, given in the engineering journals from time to time, indicate a general saving in first cost, ranging from 10 to 25 per cent in the ordinary foundation by using concrete piles instead of timber piles. In special cases, savings as high as 50 per cent are shown.

Cement, sand and stone being generally available all over the country, there is generally less probability of delay with concrete

piles than in waiting for the shipment of timber piles to the site. This time factor is often a considerable one. As our forest resources are reduced, it is increasingly difficult to get the larger sizes of timber piles, and the quality of wood is getting poorer all the time. Engineers realize how difficult it is to find a fair percentage of timber piles which fill all the requirements of the specifications, particularly when the length is 50 ft. or over. However, with reasonable care every concrete pile can be made to comply fully with the specifications, and the strength improves with age. It should be noted that although the safe allowable compression for concrete is less than for wood on the ends of the fibers, the loading of a pile depends more frequently upon the supporting capacity of the earth than on the strength of the pile itself. Concrete piles can often be driven through filled material, such as brick, stone, slag lumps, old timber crib work, sunken canal boats, etc., through which it is impossible to drive timber piles without injury, and, even where timber piles can be driven for some little depth into very hard material, concrete piles can generally be driven at least several feet further. An adequate exploration of the soil should always be made to determine the proper length of piles, whether of timber or concrete, as failure to do so leads, in the first case to waste of timber by excessive cut-offs, and in the second case to even more serious waste of time, labor and material.

In some cities[1] the use of concrete piles for the foundations of retaining walls for track elevation walls has effected a large saving by reducing the required width of new right-of-way at the high rates which had to be paid for such real estate.

Examples are cited of concrete piles showing unusual flexural strength. During the completion of a terminal pier at Brunswick, Ga., for steamships, a boat of 4800 tons displacement was the first to arrive, and in order to bring the steamer to her berth in a rapidly running tide, a 9-in. circumference hawser was fastened around the tenon of one unsupported concrete pile, about 32 ft. of which was above ground level, and hence was compelled to act as a cantilever. It successfully withstood this severe test. At another time a steamer ran into a pier owing to misunderstanding of signals in the engine room, and broke a number of pine piles, but the concrete piles successfully withstood the shock.

76. Pre-molded Piles (Patented Types).—Pre-molded piles are of several distinct types; some are patented, but most are unpatented. Among the patented types, four different examples are the Chenoweth, the Cummings pile, the Hennebique pile, and the Bignell pile.

The Chenoweth pile (Fig. 30) was one of the earliest constructed in America, being the invention of Mr. A. C. Chenoweth

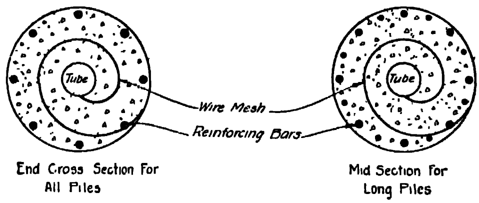


FIG. 30.—Chenoweth pile.

of Brooklyn, N. Y. It is a rolled pile and is made in a special machine. No forms are used. The reinforcement is arranged to show a spiral form in completed cross-section. This reinforcement, consisting of longitudinal bars and wire netting, is assembled on a wire platform and attached by wire clips to a mandrel or winding shaft. The concrete is spread over the reinforcement, and then by moving the platform and at the same time turning the mandrel, the pile is rolled or coiled into cylindrical form, which is compacted and shaped by adjustable rollers. The head and point are afterwards finished with concrete around projecting reinforcement, when the concrete pile is removed to a drying table. A very dry mixture of concrete must be used in making this type of pile to avoid squeezing out the water in rolling, as well as the deformation of the pile into an oval shape on the drying table. The diameter of the finished pile is generally about 15 in. Piles of this type up to 60 ft. in length have been successfully used in railroad trestles, bridge foundations, docks, etc. It is often cast with a central hole to permit the use of a water jet.

The Cummings pile distinguishes itself by the type of its reinforcement, generally of wire fabric, which is electrically welded to longitudinal rods and handled as a unit. Grooves or corrugated surfaces are sometimes molded in its surface to increase the pile surface for skin friction and also as outlet for the escape of jetting water to reduce friction in the operation of sinking. A hole or central bore cast in the concrete permits the use of a water jet, as

in the Chenoweth pile. In general, the corrugated type of pile (said to have been invented and developed by Mr. Frank B. Gilbreth) is not used as much as formerly.

The Hennebique pile is a European type and is usually constructed as a square pile without taper and with the longitudinal reinforcing bars near the four corners, which are slightly beveled or chamfered. Wire binders tie the longitudinal bars together at short intervals, and a cast-steel shoe, forming an integral part of the pile at its foot, is generally employed. An instance of the use of this type of pile is in the concrete quay at Key West, Florida. These piles were 16 to 20 in. square and 25 to 60 ft. in length, driven through marl and sand into coral rock.

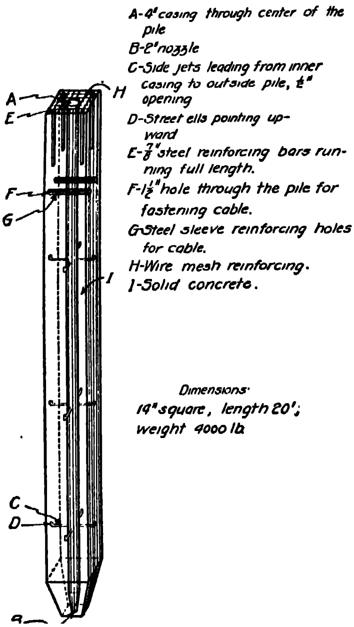


Fig. 31.—Bignell pile.

The Bignell pile (Fig. 31) is called a self-sinking pile, the principle of jetting being employed to a high degree and without the aid of a hammer.

While this type of pile has its own place as a foundation pile for structures, it has been developed and used mostly so far in a novel and successful way in the Missouri River district to connect permeable dikes in connection with river bank protection for the preservation of bottom lands, where rigid dikes have not been successful in preventing erosion during floods on account of the soft and shifting ground in the river banks and river bottom.

The pile is sunk hydraulically as one member of a system of current retards constructed upstream or alongside the river bank to be protected or built up. The top of the pile is sunk below the bed of the river to give a permanent anchorage below the scouring effect of the river bed. To this anchor is attached, by steel cables, a system of interlaced brush wood, or even large trees, which float in the water next the bank and are tied to deadmen in the bank. The brush and trees act as current retards

and cause the deposit of stream-borne sediment where it will protect the river bank instead of depositing in the channel. The general theory followed is to use the natural tendencies of such rivers to form sand bars, instead of constructing dikes and other forms of obstruction, which attempt to change the flow of the channel.

For foundation of bridges and culverts the Bignell pile is built as long as 50 ft. and 16 in. square. It consists, as for the river work already mentioned, of a reinforced concrete column with a

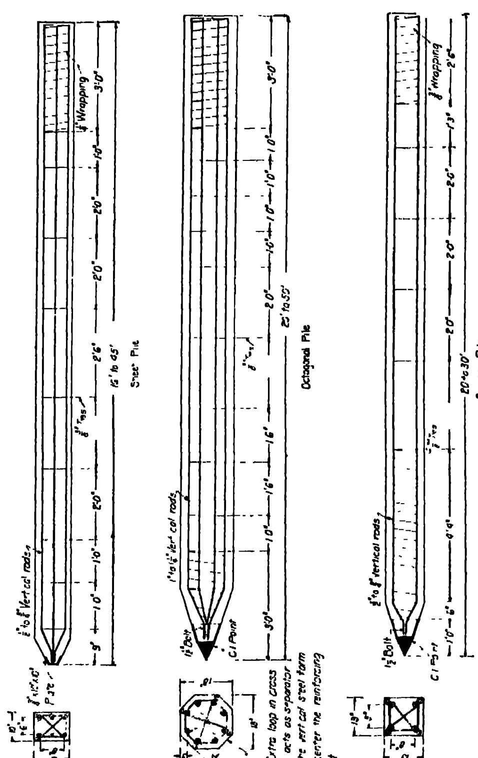


Fig. 32.—Details of piles used in ore dock construction at Cleveland.

4-in. pipe running its entire length through the center and reduced to a 2-in. outlet at the nose or point. At intervals on each of the four sides small, upturned jets are connected with this center pipe. A hose connection is made with the 4-in. pipe at the top of the pile. Water is forced through under high pressure of from 150

to 200 lb. from a duplex steam pump, and the water spurts from the nozzle, as from a fire hose, downward, and from the jets at the sides upward. The jet at the nose of the pile tends to dig downward, and the pile of its own weight sinks rapidly into the hole dug by the water pressure from the nozzle. The side jets, which are intended to form a complete film of water around

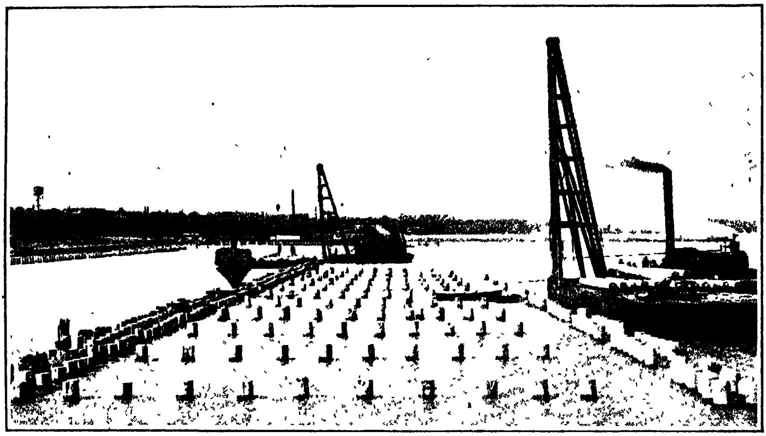


Fig. 33.—Concrete piles driven for Pennsylvania Railroad ore dock at Cleveland, Ohio.

the pile, carry away the sand and sediment dug by the point and overcome the skin-friction that is formed by the material being penetrated. While it is evident that this type of pile and sinking is most successful in sandy and silty material, it is claimed that it can also be successfully sunk through clay by the intense jetting action which is obtained. While sand or silt immediately settle around the pile when jetting ceases, it probably takes months for clay to settle close against the pile so as to develop its full value.

77. Unpatented Pre-molded Concrete Piles.—The Chicago, Burlington & Quincy Railway, through its Bridge Department, was a pioneer in the design of low reinforced concrete piles for trestles for its railroads. Concrete piles have been extensively used by that railroad, and other railroads have also adopted designs to fit their purposes.

A good example of unpatented pre-molded concrete piles is that designed by the Pennsylvania Lines for their extensive ore docks on the shore of Lake Erie at Cleveland, Ohio, as constructed

by the Great Lakes Dredge & Dock Company (see Fig. 32). Note the good lines to which these piles can be driven. They are octagonal in shape without taper to give increased friction area, as well as for ease in handling and storing. They are pointed at the foot and furnished with a cast-iron shoe which is made an integral part of the pile. The reinforcement consists of eight

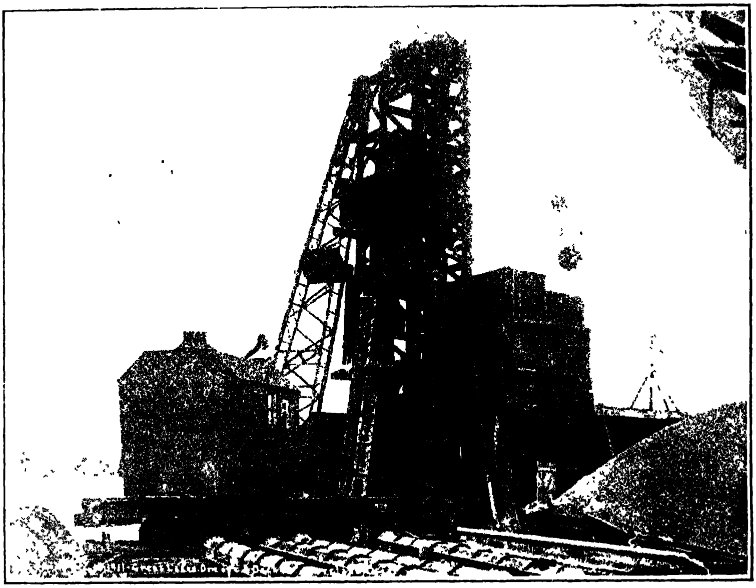


FIG. 34.—Casting concrete piles vertically in steel forms.

longitudinal rods bound together and spaced at regular intervals by tie rods. Spiral wrapping is also employed at the head and foot to reinforce the pile against shock in hard driving. Over 3500 of these piles, 18 in. in diameter and 30 to 40 ft. long, were driven for the dock foundation. The largest weighed 6 tons. Concrete sheeting, also reinforced, was driven at the channel line of the dock. All these piles were cast in vertical molds (see Fig. 34).

78. Construction of Pre-molded Piles.—In general, it should be noted that piles were first molded on horizontal forms, and that the practice of vertical molding was introduced later. These piles for the ore dock of the Pennsylvania Lines were cast vertically in steel forms, as shown in Fig. 34, and poured continuously in order to obtain dense concrete and so that the surface of the

concrete, as deposited in the piles, should be perpendicular to the direction of the load supported by the pile and to its driving. A wet mix was used, and sections weighed later showed that dense concrete was obtained particularly at the point end where it was most needed.

The hardening of the concrete was hastened by curing with live steam under cover.

After being cast vertically, they were closed at the top and placed horizontally on a floor of cross timbers, and as the season

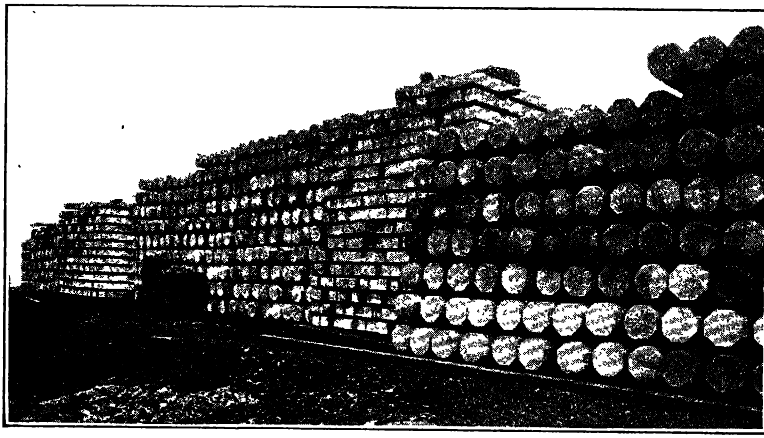


Fig. 35.—Concrete piles kept in storage 30 days before driving.

was late, artificial curing was employed, which was done by stacking up the newly made piles, covering them with canvas and introducing a steam pipe with outlet pipe discharging steam under the canavs cover, maintaining a temperature of about 80 deg. Forms were removed in 10 to 18 hr., and the concrete piles were exposed direct to the steam for 3 or 4 days afterwards, by which time they were set sufficiently to be handled by a derrick. They were then kept in storage at least 30 days before driving (see Fig. 35).

On other work concrete piles have been allowed to set 5 or 6 days in the ordinary manner and then carefully hoisted to a curing bed and stacked with wooden separator blocks between them, and subjected to live steam for 2 or 3 days. This procedure permitted them to be driven in summer within 4 or 5 days afterwards, and in winter within 10 to 12 days afterwards. Where the older method is used, namely with horizontal molds,

the side forms may be removed in 1 to 2 days after pouring, but the pile is allowed to remain on its base about a week longer. In summer it should be showered to permit complete chemical action for the setting of the cement. In very warm weather protection from the sun is advisable. After this the piles are removed and stacked to complete the seasoning, 3 to 4 weeks usually being allowed before driving.

As soon as pre-molded piles are driven, they are ready to receive their load from the superstructure above.

In best practice the reinforcement is fabricated as a unit so that it can be handled easily and quickly in the form for casting. The reinforcement unit must be held in accurate position in the molds or forms by suitable separators or hangers, so that the conditions assumed in designing the pile shall be realized in its construction.

The composition of the concrete consists almost generally of 1 part of Portland cement, 2 parts of sand and 4 parts of broken stone or gravel. Occasionally the mixture is modified to 1:2:3. In very large piles a somewhat leaner mixture is used. The size of the coarse aggregate is usually limited to ¾ in.

As sea water is found to have a somewhat deteriorating effect on reinforced concrete, if concrete piles are to be used in locations where they will be subject to direct contact with sea water, such as in exposed trestles, board walks on beaches, etc., the insistence on three simple precautions will be found of value. These are: unusual care in mixing and placing the concrete, the use of a rich mixture—say 1:1½:3—and the covering of the reinforcing steel with at least 3 in. of concrete.

The diameter of piles varies from 10 to 25 in. but seldom is below 12 in. or above 18 in. In most cases the length ranges from 20 ft. to 40 ft., although the length may vary from 8 ft. to about 75 ft., these longer lengths being used for dock construction in deep water. It is a question whether lengths less than 15 ft. should be used for foundation purposes.

79. Designing Pre-molded Piles.—The steel reinforcement of a concrete pile is intended to resist the stresses due to handling and driving the pile and to the load that may come upon it in its final position. The longitudinal bars receive their greatest stresses when the pile is lifted from a horizontal position. The pile is generally picked up near the middle, or a line may haul it by one end to the pile-driver. In the first case the pile must be strong

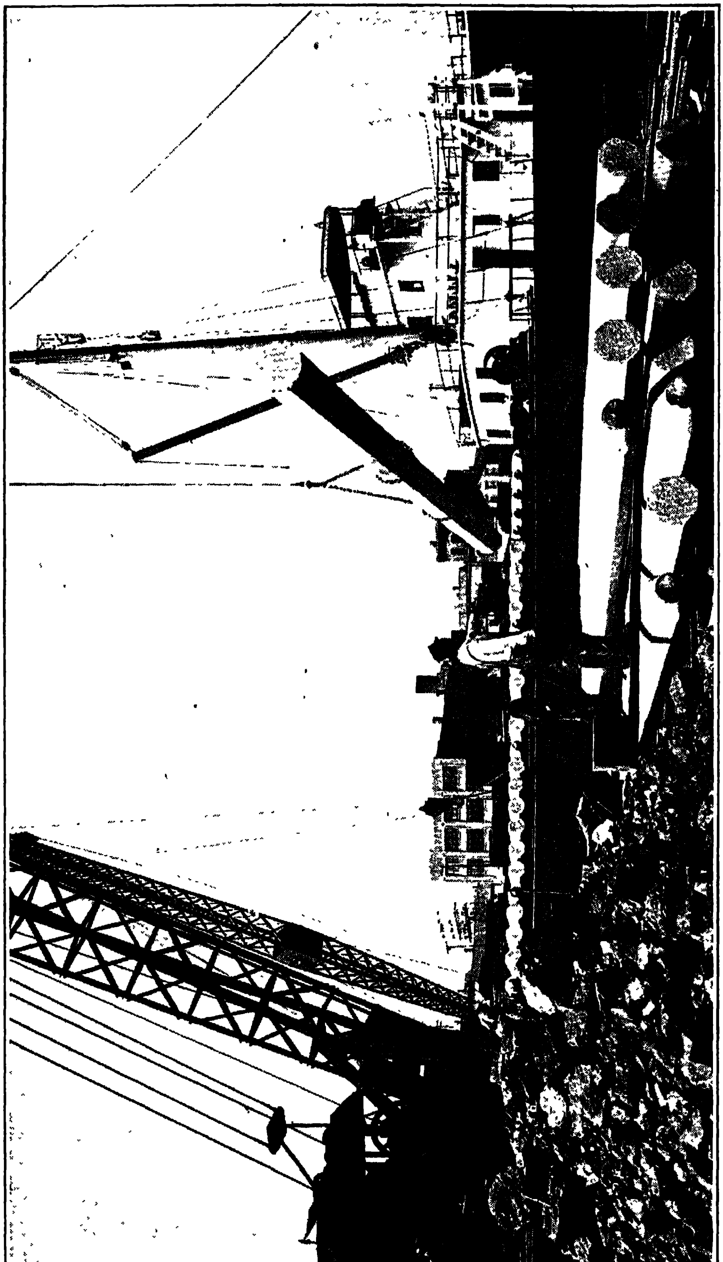


FIG. 36.—Handling concrete piles in slings.

enough to resist the flexure due to its own weight. In the latter case the pile must not only carry its own weight, but also any shock or impact from contact with obstacles. When so handled, cracks sometimes develop on the tension side, perhaps due to the reinforcing rods slipping, the concrete merely failing by compression. Exceptionally long and heavy piles are handled in slings or bridles (see Fig. 36), and extra longitudinal reinforcing is sometimes provided in the middle of the length. Some designs add 100 per cent to the weight of the pile to provide for shock due to handling. Fifty per cent is probably sufficient where proper and experienced handling is the rule.

The percentage of steel in the section area of the pile varies in practice from 0.6 to 2.8 per cent. Experiments show that hair cracks are likely to develop in handling when the reinforcing is less than 1 per cent.

The sectional area of the head must be sufficient to support in compression the safe load for which the pile is designed. The safe unit stresses used should depend upon the quality of the concrete, the percentage of reinforcement, and its arrangements, as well as the character of the loading. If a tapered pile is used, the critical section for compression is, of course, not at the head, but at some distance below. Additional allowance for hard driving in special cases is made by enlarging the sectional area, or adding extra cement, particularly at the head of the pile. When a pile is to act as a column, it is, of course, to be designed as a column. Under retaining walls or other places where the piles receive a lateral thrust, as well as a vertical load, it is necessary to use reinforced piles to resist the flexure produced, in which cases the upper portion of the pile, at least, should have a uniform section.

A minimum load of 25 tons per sq. ft. of cross-section and an additional load of 6000 lb. per sq. in. of steel reinforcement in the section is permitted by the Building Code of New York City.

In general, a concrete pile to be considered economical is supposed to carry a load of from 35 to 60 tons, depending upon its section, length, the ground conditions, etc.

The design of reinforced concrete piles is comparatively new, and engineering practice as regards safe unit stresses is not as narrowly defined as in other divisions of structural designing.

The city of Cleveland, Ohio, has used pre-molded concrete piles in the foundations of several of its large bridges across the Cuya-

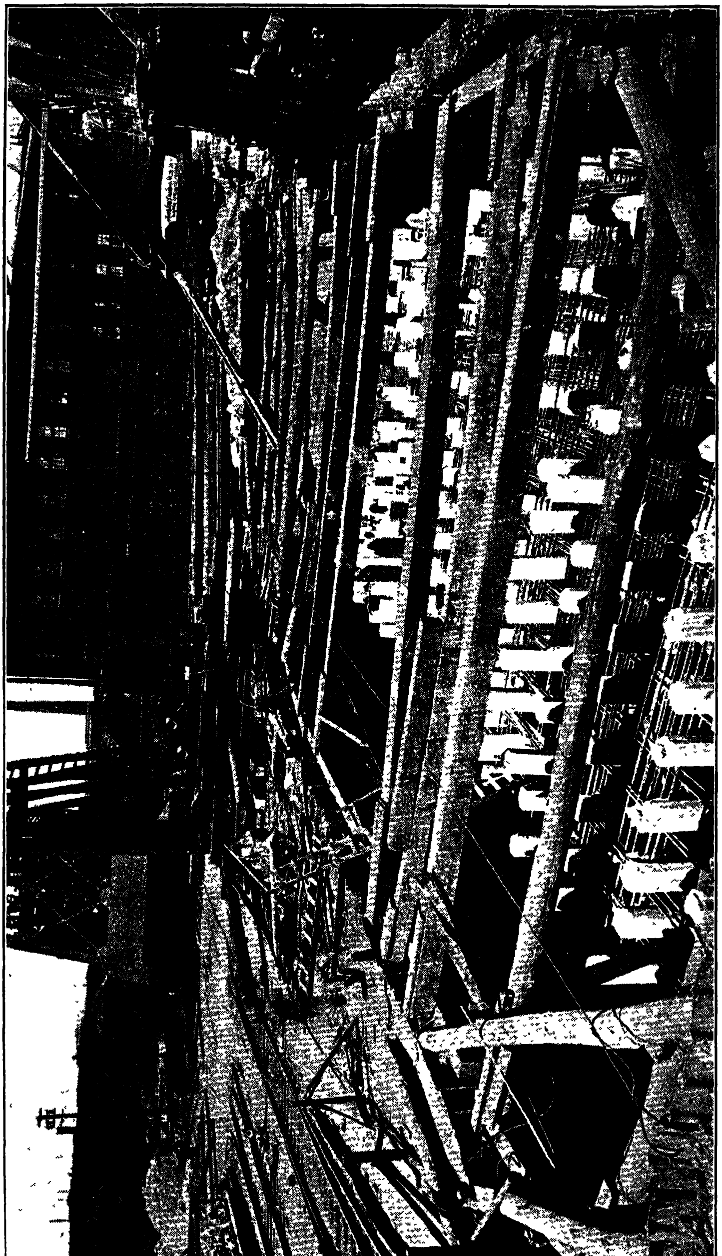


Fig. 37.—Concrete piles in foundation of Detroit-Superior bridge.

hoga River, such as the High Level or Detroit-Superior Bridge (Fig. 37), also Clark Ave. Bridge. The concrete pile specification used by the city for each bridge mentioned, and permitting the use of pre-molded or cast-in-place piles under the respective conditions stated therein, is given in Art. **87**.

Illustrated catalogues published by construction companies also give much valuable information as to sizes and disposition of reinforcement, methods of construction, and handling and driving of piles.

80. Driving of Pre-molded Concrete Piles.—The driving of concrete piles requires strong equipment on account of the great weight to be handled and the heavy hammers used. Concrete piles weighing from 2 to 4 tons are common, and on heavy construction, such as foundations for docks, etc. in deep water, long piles weighing 6 to 8 tons are employed. The leaders or towers for handling and dragging such heavy piles must have special provision made in their design for the stresses which are developed.

Wherever possible, concrete piles should be driven with the aid of the water jet, so that the duty of the hammer becomes secondary. This not merely avoids possible injury to the pile head by driving with a hammer, but also saves time and energy. The equipment and methods for the water jet, as described for timber piles, apply in general to pre-molded concrete piles. In ground which is not so advantageous for jetting, it is necessary for the hammer to do very effective work, either with or without the aid of the jet. A light hammer which answers for driving timber piles, is found uneconomical for the heavier concrete pile, and a bad tendency results, namely, to use too high a fall of hammer and expend too much of its energy in useless, and even in destructive work.

Experience in the use of both drop hammers and steam hammers for driving concrete piles demonstrates that the steam hammer drives them in less time and with less injury to the pile. However, excellent results have been obtained with the drop hammer, the heavier hammers being more efficient. Hammers weighing less than 4000 lb. have given good results, although the time required was unnecessarily long. Drop hammers weighing from 7000 to 12,000 lb. do the work much more quickly. These very large hammers are handled by three-part crucible steel cables rove over sheaves set in the hammer casting. The fall of

these long hammers is not more than 8 ft. and usually less. Examples are cited of equipment fitted with such hammers driving an average of 15 concrete piles a day to 30-ft. penetration, a maximum of 25 piles a day being obtained with one outfit. To drive concrete piles 24 in. square and 45 to 75 ft. long at Halifax, Nova Scotia, an Arnott steam hammer of the double-acting type was built of special design with a total weight of 28,000 lb., the striking part weighing 4000 lb. The stroke was 36 in. The single-acting steam hammer shown in Fig. 28, p. 152, has a total weight of 16,000 lb., the striking part weighing 7500 lb., and has been success fully used to drive thousands of concrete piles 30 to 45 ft. long for foundations such as shown in Fig. 38 for New York Central R. R. Bridge at South Chicago.

When jetting cannot be used as an aid to driving, the successful driving of pre-molded piles without injury is mainly due to the various driving caps and forms of protection used. Sometimes a hardwood block in a cast-iron cap is used on top of a concrete pile. Other types of hammers provide for mats of rope or rubber belting, or bags of sawdust under blocks to make a cushion effect, in order to avoid damaging the head of the pile. The main point is to distribute the blow uniformly over the head of the pile to prevent spalling. The tendency during the last few years is to use less protection in driving pre-molded concrete piles than in the days of their early introduction, because experience has shown that even where a water jet cannot be used with success, a well seasoned pre-molded pile usually stands a remarkable amount of pounding with the hammer, and if the hammer blow is properly controlled, that very little damage results. In driving into hard clay in one instance, the material became so compacted from driving successive piles that 5000 blows of the steam hammer were necessary to put some of the piles down 20 ft. In the few piles which were broken, the crushing extended only 18 in. below the top of the head. A test was made at the Watertown Arsenal of the upper portion (about 9 ft. long) of a pile which failed due to hard driving and probably striking a large boulder about 18 ft. below the surface. The pile had been given 735 blows with a 4700-lb. hammer, with drops varying from 18 to 30 in., and the head was badly crushed. The test calls attention to the fact that the pile failed at the small end by opening oblique and longitudinal cracks, and not at the end receiving the hammer blows. This indicates that the pile was

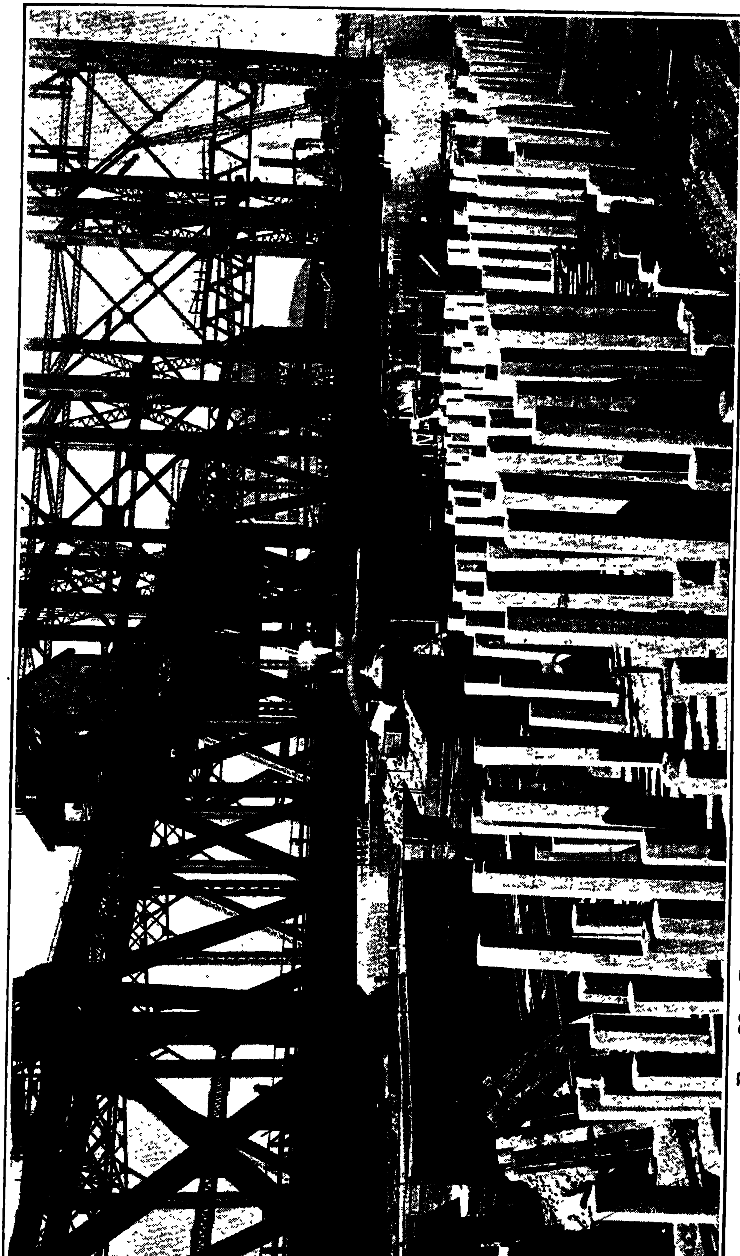


FIG. 38.—Concrete piles in foundation for New York Central bridge at South Chicago.

not materially damaged by the hammering it received, except at the immediate point of contact. The ultimate strength of the section tested was found to be considerably larger than the average strength of similar concrete columns of the same age tested with it and made of the same proportions of concrete and reinforcing. In general, the spalling of the head of a concrete pile is not considered such bad practice, as the cracked concrete is removed to expose the reinforcing rods, which are then straightened and bonded in with the concrete which is run for the footing.

Well seasoned concrete piles will stand several hundred blows of a 3000-lb. drop hammer, the drop increasing from 10 to 30 ft. as driving progresses, without appreciable injury. Comparatively green piles must be handled very carefully and the drop limited to 6 or 8 ft. Such work is slow and expensive, and it is better to season all piles thoroughly.

Concrete piles cannot be driven as rapidly as timber piles on account of the care necessary in handling the greater weights and the larger sizes of the concrete piles, as well as the greater amount of moving of the heavy driving outfit, since concrete piles are ordinarily spaced farther apart than timber piles. Roughly speaking, where the organized gait for foundations on land is 30 to 40 timber piles in a day, about 1/4 to 1/2 that many, or even less if the water jet cannot be used with effect, is considered good progress in driving concrete piles. On water work, such as dock foundations, etc., a larger number of piles can generally be driven, as the floating pile driver can move easier than a land driver, and as the comparative penetration of piles is often less considering the depth of water driven in.

For a very complete analysis of driving conditions, as observed by Sanford E. Thompson and Benjamin Fox, their paper as presented before the Boston Society of Civil Engineers on September 16th, 1908, and later printed in the Journal of Engineering Societies in January, 1909, will be found of interest. A stop watch was used in their observations, and a detailed account was kept and tabulated of both avoidable and unavoidable delays. The operations in getting ready to drive were subdivided into eight elementary operations, and the delays during driving under four heads. Their observations indicate that a pile an hour was the average gait with the aid of a water jet, and forty minutes per pile in soft ground. The piles were 30 1/2 ft. long, 14 in. square at the butt end and in general 9 in. square at the tip.

In the larger cities where building labor is expensive at the present scales of high wages, and 8 to 10 men are required in a pile-driving crew, it seldom costs less than 40 cts. per lin. ft. to drive concrete piles 30 to 45 ft. long, and often as high as $1.00 or more per lin. ft. This is for work containing several hundred piles at least, so that the expense of getting the heavy outfit to and from the site is small compared with the cost of the actual driving operations.

Concrete piles can be driven in any soil in which timber piles can be driven, and usually with much less danger of over-driving. A Chenoweth pile 13 in. in diameter and 61 ft. long was driven 8 ft. into compact gravel at Greenville, New York Harbor, where oak piles could not be driven.

It sometimes is found necessary to cut off the head of a concrete pile which cannot be driven to full depth. In such cases the use of a track chisel and a heavy hammer will dispose of the concrete, as usually it is not more than one or two months old and is not as difficult to break as older concrete. A hack saw will cut off the reinforcing bars, or the bars may usually be bent with a pipe and encased in the concrete footing, if desired.

81. Cast-in-place Piles.—The cast-in-place concrete pile is one that is constructed in its permanent position in a hole in the ground prepared for the purpose. The cast-in-place piles of all types are patented, the characteristic feature of each type being largely the method of construction and the appliances used for the construction.

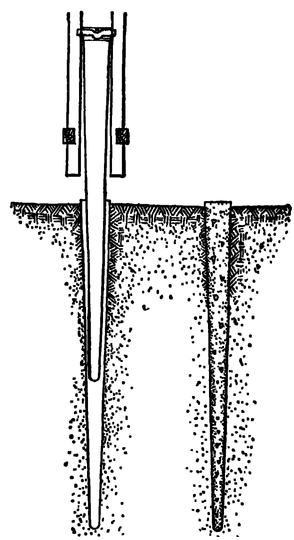


FIG. 39.—Raymond pile core collapsed and partly withdrawn. Completed Raymond pile without reinforcement.

The Raymond pile (see Fig. 39) is made by driving a tapering casing or shell of sheet steel into the ground by means of a steel core or mandrel, which supports the steel during driving, but which is collapsed and withdrawn after the required penetration is reached, so that the casing can be filled with concrete. On account of the con-

siderable weight of the core, the pile-driver used is generally of very heavy construction and fitted with a heavy steam hammer. The casing or shell is usually 18 to 20 gage sheet steel and is made up in conical sections about 8 ft. long, which telescope for the purposes of shipment, but which overlap tightly when in place, this placing consisting of slipping the sections over the core in proper succession. The short section at the bottom of the pile is called the "boot" and is usually made of heavier steel to stand the cutting effect of stone or other obstructions encountered in driving. The object of the casing is to act as a mold or form that shall preserve its shape until the concrete is set, and to prevent the earth and water driven through from mixing with the concrete. The interior of the casing may be inspected before placing the concrete by means of an electric bulb lowered into the casing by long wires.

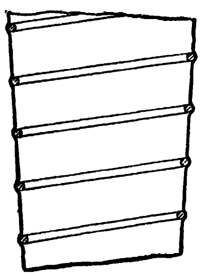


FIG. 40.

In earlier construction the thin casings were sometimes collapsed by hydrostatic pressure, necessitating driving several shells, one inside the other. This feature was later improved by reinforcing the casings with ¼-in. wire used as a spiral, which materially increases the strength (see Fig. 40). The concrete used varies from a 1:2:4 to a 1:3:5 mixture to meet the particular conditions of each foundation. A rather wet mixture is favored. The coarse aggregate is usually ¾-in. to 1½-in. stone or gravel. Longitudinal reinforcement is occasionally used, but not in general, although the inserting of short rods is often done to bond the tops of the piles to concrete footings. It is evident that the casing with its spiral wire wrapping the concrete, acts as lateral reinforcement. The avoidance of interior reinforcement renders it easy to place concrete in the smooth casing so as to obtain good concrete without voids. Raymond piles in standard sizes have a diameter of 20 in. at the head for lengths of 20 to 30 ft., and 18 in. for lengths of 35 to 40 ft. The tip is usually 6 in. for the shorter piles and 8 in. for the longer ones. As the considerable taper of the Raymond pile is the great advantage claimed for it, it is evident that in piles longer than 40 ft., the tip of pile will be quite small, and, therefore, this type of pile has a very limited use beyond a length of about 40 ft.

The Raymond type of pile is successfully employed in America, mostly for foundations of buildings. It is evident it cannot be used to any extent in marine work, where the pre-molded pile naturally holds sway. The special advantages claimed for it over other concrete piles are: (1) the economy due to the large taper, permitting a reduction in the length of piles; (2) the speed with which piles can be placed; (3) the opportunity afforded for testing of the bearing power of every pile by the average penetration of the steel core under the final blows of the hammer; (4) the ability to drive through very hard material due to the steel core; (5) the facility for the inspection of the form in which the concrete is placed; and (6) it is the only type of cast-in-place pile which can be used to project above the ground without the use of special forms at increased cost.

The principal disadvantage of this type of pile is that it does not appear to be developed for jetting methods. It is almost always driven with a hammer in fine sand and similar ground, where conditions are ideal for the use of the water jet. This pounding away in sand, etc. not only consumes time and energy, but the vibration set up very probably causes injury to the green concrete in the adjacent piles.

Other types of cast-in-place piles are the Simplex and the Pedestal piles.

The Simplex pile, introduced in 1903, is constructed by driving into the ground a steel pipe, generally 16 in. in diameter and ¾-in. thick, the pipe being fitted at the driving point with a special shoe or jaw either of cast-iron or concrete, which completely closes the bottom; the hole thus made is filled with concrete as the pipe is withdrawn (see Fig. 41). The pipe must be extra heavy, as long as the pile to be formed, and the piledriving equipment must be strong enough to pull out the pipe. The shoe remains in place, and hence a new one is needed for each pile. To save the cost of a point for each pile, in very firm earth a device called an alligator point is used, which opens automatically when the pipe is pulled out and

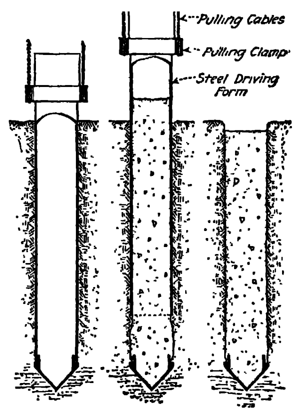


Fig. 41.—Method of constructing Simplex concrete piles.

permits the concrete to flow through it. In some work the pipe is filled with concrete, at the same time slowly withdrawing it. In other cases, as each batch of concrete is dumped in the hole, it is rammed to force it against the surrounding earth, in order to completely fill the hole. This increases the diameter of the pile and increases the bearing area.

Fairly wet concrete of a 1:2:4 mixture is generally used. Stone or gravel of ¾-in. size is usually employed. The advantage claimed for this type of pile is that the concrete is forced into the irregularities of the compressed earth, giving a frictional resistance greater than for any other pile of equal proportions. However, the compressed earth sometimes becomes a part of the pile section and changes the frictional surface to a more regular form. In very soft ground where the earth does not hold its form as the pipe is withdrawn, this type of pile cannot be used without modification. In such cases a light casing of smaller diameter is generally used by lowering it into the hole as soon as the first batch of concrete is placed. When this form is filled, the driven pipe is withdrawn. It is evident this leaves some voids outside the light sheet metal casing, which will fill only by the adjustment of the surrounding earth.

Piles as long as 45 to 48 ft. have been placed by the Simplex method, the practical limit being the ability of the equipment to pull out the pipe. In firm, dry soils it is claimed to be the cheapest method of installing concrete piles. In ground that has any tendency to flow, there is always the question as to what extent the strength of the pile may be reduced by an admixture of earth, as it is impossible to inspect the integrity of the pile during construction.

The Pedestal pile is a modification of the Simplex pile by the addition of a base or bulb-shaped pedestal at its foot. It is the invention of Mr. Hunley Abbott. The intent of its form of construction is to take greater advantage than the ordinary form of piles do, of the higher bearing capacity generally existing in the lower strata of earth. This increasing of the bearing area at the foot doubtless was suggested by the older form of metallic screw and disk piles whose action it imitates (see Fig. 42).

Equipment similar to that used for the Simplex pile is used to construct the Pedestal pile, except that a bottom shoe or jaw is not used, and that a steel core is added, which fits inside the pipe with its lower end projecting 4 or 5 ft. below the pipe,

and with its upper end enlarged into a head which fits over the pipe for driving. The pipe is usually about 16 in. in diameter and made of 3/8-in. metal. The pipe and core are first driven into the ground as a unit. The core is then withdrawn and a batch of concrete is dumped into the pipe, and the core is then used as a rammer to enlarge the hole below the pipe and to push aside the concrete laterally. When the concrete base is given the required volume by this method, the pipe is filled with concrete and gradually pulled out. A 1:2:4 mix of concrete is generally

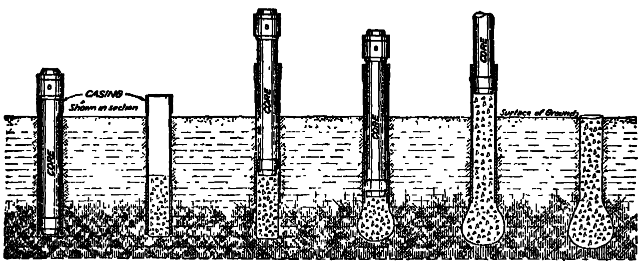


FIG. 42.—The process of forming a pedestal pile.

used. The finished pipe is usually about 17 in. in diameter if a 16-in. pipe is used, and the base is about 3 ft. in diameter and contains about half a yard or a little more of concrete, depending upon the nature of the ground. Should the earth for any reason resist unequally on opposite sides of the hole, the form of base resulting would make its reaction eccentric.

82. Objections to Cast-in-place Piles.—The chief objection to all cast-in-place piles is based upon the probability of injury to the green concrete from the back pressure set up by driving the forms for adjacent piles.

Even if the casing left in the hole, or the weight of the concrete without the casing is able to resist an outside pressure until the cement is set, it is probable that the green concrete will be injured by the vibration and additional earth pressure from driving adjacent piles after the cement has started to set and before the setting has been completed.

Numerous tests to determine conditions have been made by excavating. In one example failure was due to fluid soil penetrating between batches of concrete, separating the pile into sections about 5 ft. long and destroying its value as a pile. In other cases piles were found bent out of line. In still other cases the section areas were reduced from 20 to 100 per cent. In one

case, as ascertained later by analysis, the cement failed to set due to chemical constituents in the ground water. It was also found that the liability of green concrete to suffer injury from adjacent driving is increased when hard and soft strata alternate. Unless protected by a casing there is always danger of some of the cement being washed out by underground flowing water, or contrawise, absorbent earth may deprive cement of some of the water it requires to set completely.

The construction of cast-in-place piles requires more careful supervision than for pre-molded piles, in order to get good results, on account of the manner in which concrete is deposited and the surrounding conditions, which preclude inspection of the pile after the concrete is in place. When it is found necessary to put reinforcement in a cast-in-place pile for its whole length, it should be fabricated as a unit and set carefully in proper position, instead of placing bars separately, in order to insure their occupying the specified positions in the finished pile.

83. Composite Types and Combination Piles.—Hollow premolded piles are sometimes driven and filled with lean concrete after driving. Such hollow piles are generally of large sizes to obtain economy, and must be carefully handled by special slings or bridles to avoid bending stresses. A type of concrete pile called the Peerless, has also been used. It consists of a reinforced concrete shell made in sections and slipped into a steel driving pipe, both of which bear on a pointed cast-iron shoe, which is left in the ground. The steel pipe protects the concrete shell from stresses due to driving, and when the steel pipe is pulled out, the concrete shell is inspected and filled with tremie concrete.

Combination piles have been made for foundations of docks on the Pacific Coast by driving a hollow, reinforced, concrete pile 4 in. thick and 24 in. in diameter over the top of a wooden pile which had already been driven into the channel bottom, so that the head of the wooden pile projected a few feet above the mud line. When a good bearing was obtained on the bottom with a concrete pile, the inside was pumped out to remove the mud and water, and the hollow space was filled with concrete. It is feasible to use such combination piles for foundations on land, and they are cheaper than very long concrete piles.

At times the durability of concrete piles is combined with the lesser weight and cost of timber piles placing concrete piles on top of timber piles which are driven below ground water level

to resist decay. Such a combination of a cast-in-place concrete pile with a wooden pile tip and aggregating in total length 60 to 80 ft., was used in the foundation of the ore docks for the Henry Ford Company at River Rouge, Detroit, a few years ago on account of the extremely soft material encountered. The wooden pile was put in a lathe and its head turned down, for a length of about 18 in., to about 7 or 8 in. diameter, so as to fit the end of a metal casing for a Raymond concrete pile. The core was put in the Raymond casing, and the long pile thus assembled was driven down, the core being then removed and the casing filled with concrete, similar to the usual cast-in-place piles of this type.

84. Choice of Type of Concrete Piles.—In any case where it has been decided that the use of concrete piles is more economical than timber piles, durability also being given full consideration, and the conditions at the site and the nature of the ground are known, the question then remaining is to determine the type of pile especially adapted to the conditions in hand, so that adequate strength may be secured for the proposed structure at the most reasonable cost. As each type of concrete pile has some distinctive advantages which are adapted more or less closely to certain conditions of ground where piles are necessary, to use a type of pile under conditions which are not favorable to it, means either lack of economy or less security, or both. Some types cover a wider range of conditions than others, and the engineer's duty is to make a special study of each situation.

For supporting a structure above open water, as in docks, piers, wharves, pile trestles, etc., in addition to acting as columns, the piles are also required to resist flexure. Pre-molded or pre-cast piles are the only ones adapted for this service. They should be made without taper, at least for that part of the length which is not in the ground. Taper is valuable in sand where the supporting power is due almost entirely to friction, and if the piles are to penetrate sand, that portion may be tapered. However, if the sand is subject to scour, or if sufficient total penetration and friction can be obtained without taper. then a pile with uniform cross-section should be employed.

The pre-molded pile has special advantages in ordinary sand and quicksand, or in such combinations of sand with gravel or clay as produce porous masses, because in such cases the water jet can be used successfully. When a pile is to be driven through

soft material into a harder stratum, and so that it is expected to act as a column, it must be reinforced, and frequently the pre-molded pile is the only one that can be so used.

It is important to remember that after plain concrete piles are driven in some kinds of stiff clay, if the adjacent ground should be heavily loaded, lateral pressure will be developed, causing serious bending moments which piles without longitudinal reinforcement may be unable to resist safely.

Conditions are sometimes met with where deep beds of clay require pile foundations because the upper strata becomes soft during the flood season, while during the dry season they are favorable for construction work and the clay is so hard that it is impracticable to drive piles into it. In such cases the problem often may be worked out satisfactorily by excavating a hole of proper diameter in the hard clay by means of an earth auger and driving a pre-molded pile well into it, so as to fill the hole so completely that the surface water will not follow down the pile and impair its value. Where investigation of the sub-surface shows a stratum of quicksand or other soft material, which indicates that it will not retain its form until the pressure of the concrete can resist the external pressure, then no cast-in-place pile should be employed unless it leaves a casing in the ground which will retain its form until the concrete sets. The casing should have a uniform diameter, instead of a taper, so as to secure a larger bearing area at the foot. If the upper strata are of such nature as to retain their form temporarily until the concrete is in place, the type of pile may be used in which the pipe is gradually withdrawn. If a considerable part of the load is to come on a bottom stratum which is not well defined on its upper surface, the enlarging of the base of the pile to increase the bearing surface is often desirable, as is done in the Pedestal pile method. But as the method of making the Pedestal pile requires the ground adjacent to the base to be displaced by the ramming of the concrete, if the ground material is not homogeneous, the base may be unsymmetrical with reference to the vertical axis, and a dangerous stress may be produced in the stem of the pile due to the eccentric reaction of the pile column. In general, whenever the load is carried mainly at the foot of the pile, the pile should be reinforced unless the upper strata afford good lateral support. Also there should be a limiting ratio of length of pile to its diameter.

All cast-in-place piles require precautions as to the order in which the piles are placed, so that no core or pipe is driven for another pile within a prescribed distance of a completed pile during the setting of its cement.

Where the nature of the ground is tough and leathery so as to cause upheaval when adjacent piles are driven, it would be disastrous to use some types of cast-in-place piles. As far as form is concerned, any piles used should be without taper. In ground that is compressible but not soft in the upper layers and gradually increases in density downward, almost any of the different types of piles may be used with proper precautions, but without taper, so that advantage may be taken of greater bearing value at the foot, as well as to obtain the greater frictional resistance of the lower surface of the pile.

To go a little further with this general illustration—if the ground is soft to a considerable depth, but the density increases slowly with the depth so that the bearing power depends almost entirely on skin-friction, two factors will decide whether a tapered or an untapered pile is to be used. As the pile with uniform section has a slightly larger superficial area for a given volume, it has the additional advantage of having a larger proportion of its surface in the lower portion of the pile, where friction is greater. On the other hand, the tapered pile has larger sectional area of concrete at the top to transmit the load, and that factor may govern in some cases. In general, as the load is gradually transferred to the surrounding earth in passing downward through the pile, the decreasing sectional area of the tapered pile makes it conform more closely to one of universal strength throughout.

The pre-molded pile is favored by many engineers over the cast-in-place types, because the concrete can be made, seasoned and inspected thoroughly before any driving is commenced.

In some of the larger cities, building regulations take cognizance of the superiority of pre-molded piles by allowing a higher safe load per square inch of cross-section of pile. For instance, in New York City, in about the year 1916, a safe load of 500 lb. per sq. in. of cross-section was established as the allowance for pre-molded piles and 350 lb. per sq. in. for cast-in-place piles. This is about 43 per cent more safe load allowed for pre-molded piles per unit of sectional area than for cast-in-place piles.

It must be kept in mind, however, with all that has been said in favor of concrete piles as a class, that there are some limitations

which still leave a field for timber piles. For example, in marshy land, timber pile trestles are found superior to concrete trestles, due to less settlement, etc., and a combination of the timber pile with concrete top can often be used to reduce loads and also costs.

85. Driving and Loading Test Piles.—No standard practice has yet been developed with concrete piles as to allowable settlement under test for given loadings. This is due to the comparatively short time concrete piles have been employed. However, a few examples from specifications will indicate the general trend of practice.

In the specifications for concrete piles for the foundations of the Detroit-Superior Bridge at Cleveland (see Art. **87***b*), at least two piles were required to be tested in each pier. Each pile, whether pre-molded or cast-in-place, was required to sustain a test load of 60 tons with no settlement more than ¼ in. at the expiration of one week. This was in material ranging from sand to stiff blue clay. The price bid for each test pile called for furnishing all labor and material to install a balanced load of at least 70 tons on each test pile.

In the specifications by the same city for the foundations of Clark Avenue Bridge (see also Art. **87***a*), it required each concrete pile to be driven to refusal or until the resistance to driving was such that the last 25 blows of the specified hammer on the solid, unbroomed head of a timber pile, or the last 35 blows on a solid, hardwood block on the sound head of a concrete pile or steel shell, would not cause a penetration of more than 1 ft. If a follower was used or test piles driven, these requirements for resistance to driving were to be modified as the engineers should determine. The hammer specified was a steam hammer, equipped with a McDermid base or equivalent, and with the striking part weighing at least 5000 lb., and falling 3½ ft.

Fig. 43 shows a test pile on this work in the year 1913, loaded with 60 tons of steel sheeting and pig iron. The pile was 40 ft. long, octagonal in section, 16 in. in diameter. Penetration 30 ft. Total number of blows 2339. For last foot of penetration 328 blows. Settlement 0.023 ft., or slightly over ¼ in. The rope mat used to protect the head of the pile in driving can be seen in the left foreground of photo.

In a specification for a building foundation, a safe load of 25 tons was assumed for each pile, and not more than ¼-in. settle-

ment was allowed on any one of six test piles under a test load of 40 tons each. Piles were used varying from 30 to 40 ft. in length. The bottom was soft blue clay in irregular strata, alternating with strata of stiff material. The overlying material was sandy soil.

The Building Code of the City of Chicago states that the allowable load on concrete piles shall be taken as one-half of the load

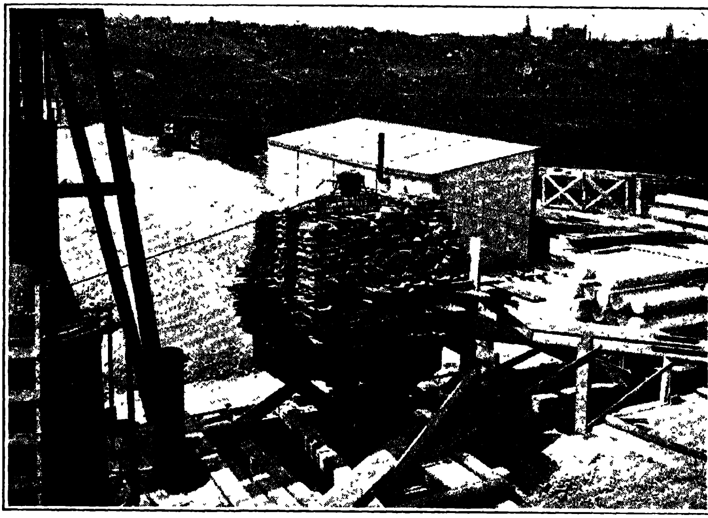


FIG. 43.—Test load on concrete pile.

which shows no settlement for 24 hr.; and the total settlement is not to exceed 0.01 in. per ton of test load. The allowable compression is not to exceed 400 lb. per sq. in. at a section of 6 ft. from the surface of the ground in immediate contact with the pile.

This code, as revised in 1917, has special requirements for cast-in-place piles. The test loads are to be applied on at least two piles in different locations designated by the Building Commissioner, not less than three piles being driven at each such location. The pile to be loaded is placed first; within 6 hr. a second pile, and within 20 to 24 hr. a third pile, are to be placed at distances from the first not to exceed twice the greatest diameter of the pile, measurement being made from center to center.

The tests are not to be made until ten days after the placing of the piles which are to be loaded. The remainder of the test

is to be the same as for pre-molded piles, which type is allowed to be tested as soon as practicable after driving.

At the Charleston, S. C. Navy Yard a pier was built, using pre-molded, reinforced, concrete piles, each contractor submitting his own design for concrete piles. The length of piles used was 55 ft. and they were 18 in. square except for a few feet of taper at the point. The material driven through was marl, a sticky yellow clay with about 15 per cent sand in it. A 4500-lb. drop hammer was employed, but the driving was so hard, "churning" had to be resorted to—that is, raising the pile with the hammer on it, about 2 ft. and then dropping it; also the water jet. The last few inches of driving were done by shutting off the water jet and using the hammer without any churning. The test load required was 20 tons to the pile on any pile chosen by the Government after driving. Under a load of 30 tons for 48 hr., no settlement whatever was found.

Fig. 33, p. 172, shows a test pile loaded during the driving of concrete piles for the Pennsylvania Company's ore dock at Cleveland, referred to on p. 172.

Pre-molded piles driven with heavy drop hammers often show good results under test. In one case about 27 ft. of penetration was obtained through silt, sand and gravel, using a 7000-lb. hammer. A test load of 63 tons caused a settlement of but ⅛ in. in two weeks. In another case a total penetration of 30 ft. was obtained using a hammer weighing 12,000 lb., and when loaded with a weight of 72 tons the pile showed no settlement at the end of six months.

About the year 1908, pre-molded concrete piles were driven for the foundation of a power station at Oakland, California. The ground, as shown by test borings, was hard yellow clay, overlaid by 8 to 12 ft. of mud and cinder fill. The work was situated only a few hundred feet from the Oakland estuary, and as the low water level was about 10 ft. below the ground level, wooden piles were out of the question even if they could be driven into the hard material. A spread foundation on the hard, yellow clay would have necessitated extensive trench work, costly pumping, and a great waste of masonry. Concrete piles were adopted somewhat against the advice of local engineers and contractors. The piles used were 12 in. square and from 20 to 26 ft. long, and were reinforced. A steam hammer, with striking part weighing 5500 lb. was used, giving about 60 blows

to the minute. The piles drove so rapidly through the fill that very little hammering was necessary. After that it took from 200 to 250 blows to drive the pile to full depth. The penetration of the piles, while driving into the hard, yellow clay, was nearly uniformly 1 in. per blow. The piles were designed for 30 tons load for the boiler room and the building structure proper, and 25 tons for the turbines. A test load of 43 tons on the first pile driven produced a settlement of 3/16 in. after 24 hr., which load, after being increased to 50 tons, produced a settlement of 2½ in. As all the other piles drove much harder than the first pile, it was considered that 50 tons was a safe measure of test carrying capacity.

While the phenomena of pile-driving give a fair measure of the bearing power of most conditions of ground, there are exceptions for which the engineer should constantly be on the watch. As some moist clays are almost incompressible, but are at the same time plastic, piles driven into such ground often displace the material and force the surface upward in other places. Unless levels are taken, this movement is apt to escape observation. In such cases the loading of test piles will reveal the conditions. Sometimes tests and levels of this kind lead to a change in the type of foundation adopted.

86. Prices of Concrete Piles.—Concrete piles like timber piles and all other construction items cost considerably more now than before the World War. In the year 1909 the price of $1.00 per lin. ft. was quoted for Chenoweth piles delivered alongside the work ready for driving; the piles specified were 34 ft. long and 16 in. in diameter, for a railroad crossing over a drainage canal in Evanston, Ill. In the same year three docks were under construction in Havana, Cuba, employing the same type of pile, 60 ft. in length. The total cost for manufacturing and driving these piles was given as about $1.50 per lin. ft. Raymond type of cast-in-place piles were installed in building foundations during the same period at prices varying from $1.50 to $2.00 per lin. ft., according to conditions.

In the year 1912 about 1000 reinforced pre-molded piles 13 in. square and 16 ft. long were furnished on cars at the manufacturer's plant for track elevation work at Cleveland, at the rate of 75 cts. per lin. ft., the railroad company furnishing the cement for the piles. Each pile contained 7/10 yd. of concrete and weighed 1.4 tons. The materials entering into the finished pile

were $\frac{1}{3}$ cu. yd. of sand and $\frac{2}{3}$ cu. yd. of gravel, 114 lb. of reinforcing steel and 70 lb. for pile point, bolts, etc., and about $4\frac{1}{2}$ bags of cement.

In the years 1910 to 1914, inclusive, the heavier types of concrete piles 16 to 18 in. in diameter and 30 to 45 ft. long, such as were used in the Pennsylvania Ore Dock at Cleveland and in the Clark Ave. and Detroit-Superior Bridges at Cleveland, were quoted at prices ranging from 85 cts. to $1.25 per lin. ft. on board cars at the manufacturing plant. This was for octagonal piles, but piles square in section, with less heavy reinforcing and simpler in construction, were quoted at lower prices.

The corresponding present day prices for heavy concrete piles of octagonal section vary, according to the reinforcement and pile points used, from $1.50 to $2.50 per lin. ft. on cars at site of manufacture.

Cast-in-place piles of the Raymond type are being installed in building foundations at prices varying from about $2.00 to $3.00 per lin. ft. of completed pile in place, depending upon the length of the piles, the number of piles in the foundation, ground conditions, the local labor situation, etc. Pre-molded piles, 12 to 14 in. square and simple in construction, are being furnished and driven at prices varying from $1.25 to $2.00 per ft. of pile.

The cost of manufacturing pre-molded concrete piles usually can be closely estimated for given conditions by experienced manufacturers, but the cost of driving, for the reasons stated in preceding sections, is not susceptible of such close estimating.

87. Concrete Pile Specifications.

87*a*. For Clark Ave. Bridge, Cleveland.—Concrete piles shall be made of 1:2:4 concrete and may be either pre-molded and driven, or cast-in-place. The design shall be satisfactory to the Engineer, and materials and workmanship shall fully meet the requirements hereinafter specified for concrete structures. The diameter or least dimension of piles having constant cross-section shall not be less than 14 in. and the diameter or least dimension of taper piles shall not be less than 8 in. at the tip and not less than 14 in. at mid-length. Pre-molded piles shall be cast in vertical position, cast at least 45 days before driving, and numbered and dated as cast. They shall have sufficient steel reinforcement, not less than $1\frac{1}{2}$ per cent longitudinally and not less than 15 per cent peripheral—to permit rough handling and driving without injury. They shall be straight, true to section, free from soft spots, cracks, honeycomb or other defects. The pitch or spacing of the peripheral reinforcement throughout the body of the pile shall not exceed 8 diameters of the rod for a length of twice the diameter of the pile. The piles shall be shod with approved steel points

rigidly attached to the longitudinal reinforcement. The longitudinal and peripheral reinforcement shall be formed into a unit. Heads shall be enriched by an excess of 25 per cent of cement. Cast-in-place piles shall have permanent steel shells, which shall be water-tight, and of such construction as to withstand the driving and the soil pressure. Piles premolded or cast-in-place for foundations of abutments, retaining walls, or arches, shall be properly reinforced for lateral forces.

Each pile shall be driven to refusal or until the resistance of driving is such that the last 25 blows of the specified hammer on the solid, unbroomed head of a timber pile, or the last 35 blows on a solid, hard-wood block on the sound head of a concrete pile or steel shell, will not cause a penetration of more than 1 ft. The length of piles in all cases must be sufficient to fully meet this requirement. If a follower be used, or test piles driven, the requirement for resistance to driving shall be modified as the Engineer shall determine.

Piles shall be driven by a steam drop hammer having striking parts of at least 5000-lb. weight and falling 3½ ft. The hammer shall be equipped with a McDermid base, or equivalent. For driving to an appreciable distance below the driver, the piles shall be driven by a follower through a strong steel tube, rigidly held in position. The piles shall be driven in position shown on plans; bearing piles shall be driven vertically and batter piles on the required inclination. If any pile develops the desired resistance to driving before being driven to full depth, the driving shall continue, if required by the Engineer, until the required penetration is attained.

After the piles have been driven the heads shall be cut off at correct elevation.

87b. For Detroit-Superior Bridge, Cleveland.—Concrete piles may be either cast piles or molded in place. Cast piles shall be reinforced with vertical bars of a total area of at least 1 per cent of the mean sectional area of the pile. The head of the pile shall be composed of a mixture of concrete 10 per cent richer than the main body of the pile. The entire pile shall be hooped with suitable reinforcement. For a distance of 3 ft. from the top and bottom of the pile extra hooping shall be provided to withstand the driving. Concrete for all piles shall be composed of one part cement, two parts sand and four parts of 1-in. broken stone as specified. Cast piles shall have suitable driving points of cast-iron.

If piles are molded in place, they shall be of such design that a reinforced steel shell, ready for concreting, shall be left in place. The Engineer may require that no shell shall be filled with concrete until the surrounding holes are driven. The driven shell shall be capable of withstanding back pressure. All piles molded in place shall have vertical reinforcement as specified for cast piles.

The Contractor shall bid a unit price for testing piles. At least two piles will be tested in each pier. These tests will be on single piles and, whether cast or molded piles, they shall be capable of sustaining a load of 60 tons with no settlement more than ¼ in. at the expiration of one week.

The length of piles will be determined by the resistance to driving and the results of the tests. Piles shall be jetted through sand and gravel, should the use of the jet be deemed necessary by the Engineer.

were 1/3 cu. yd. of sand and 2/3 cu. yd. of gravel, 114 lb. of reinforcing steel and 70 lb. for pile point, bolts, etc., and about 4½ bags of cement.

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The corresponding present day prices for heavy concrete piles of octagonal section vary, according to the reinforcement and pile points used, from $1.50 to $2.50 per lin. ft. on cars at site of manufacture.

Cast-in-place piles of the Raymond type are being installed in building foundations at prices varying from about $2.00 to $3.00 per lin. ft. of completed pile in place, depending upon the length of the piles, the number of piles in the foundation, ground conditions, the local labor situation, etc. Pre-molded piles, 12 to 14 in. square and simple in construction, are being furnished and driven at prices varying from $1.25 to $2.00 per ft. of pile.

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rigidly attached to the longitudinal reinforcement. The longitudinal and peripheral reinforcement shall be formed into a unit. Heads shall be enriched by an excess of 25 per cent of cement. Cast-in-place piles shall have permanent steel shells, which shall be water-tight, and of such construction as to withstand the driving and the soil pressure. Piles premolded or cast-in-place for foundations of abutments, retaining walls, or arches, shall be properly reinforced for lateral forces.

Each pile shall be driven to refusal or until the resistance of driving is such that the last 25 blows of the specified hammer on the solid, unbroomed head of a timber pile, or the last 35 blows on a solid, hard-wood block on the sound head of a concrete pile or steel shell, will not cause a penetration of more than 1 ft. The length of piles in all cases must be sufficient to fully meet this requirement. If a follower be used, or test piles driven, the requirement for resistance to driving shall be modified as the Engineer shall determine.

Piles shall be driven by a steam drop hammer having striking parts of at least 5000-lb. weight and falling 3½ ft. The hammer shall be equipped with a McDermid base, or equivalent. For driving to an appreciable distance below the driver, the piles shall be driven by a follower through a strong steel tube, rigidly held in position. The piles shall be driven in position shown on plans; bearing piles shall be driven vertically and batter piles on the required inclination. If any pile develops the desired resistance to driving before being driven to full depth, the driving shall continue, if required by the Engineer, until the required penetration is attained.

After the piles have been driven the heads shall be cut off at correct elevation.

87b. For Detroit-Superior Bridge, Cleveland.—Concrete piles may be either cast piles or molded in place. Cast piles shall be reinforced with vertical bars of a total area of at least 1 per cent of the mean sectional area of the pile. The head of the pile shall be composed of a mixture of concrete 10 per cent richer than the main body of the pile. The entire pile shall be hooped with suitable reinforcement. For a distance of 3 ft. from the top and bottom of the pile extra hooping shall be provided to withstand the driving. Concrete for all piles shall be composed of one part cement, two parts sand and four parts of 1-in. broken stone as specified. Cast piles shall have suitable driving points of cast-iron.

If piles are molded in place, they shall be of such design that a reinforced steel shell, ready for concreting, shall be left in place. The Engineer may require that no shell shall be filled with concrete until the surrounding holes are driven. The driven shell shall be capable of withstanding back pressure. All piles molded in place shall have vertical reinforcement as specified for cast piles.

The Contractor shall bid a unit price for testing piles. At least two piles will be tested in each pier. These tests will be on single piles and, whether cast or molded piles, they shall be capable of sustaining a load of 60 tons with no settlement more than ¼ in. at the expiration of one week.

The length of piles will be determined by the resistance to driving and the results of the tests. Piles shall be jetted through sand and gravel, should the use of the jet be deemed necessary by the Engineer.

Should the driving of any piles cause previously driven piles to rise the vertically displaced piles shall be driven to their original elevation.

Should any concrete pile fail to develop sufficient resistance to carry the specified load, the piles must be withdrawn and longer piles substituted, until the proper length to resist the specified load be determined by the Engineer. All piles shall be driven under observation of the Engineer, or his representatives, and accurate data obtained on the penetration. The data so obtained will be used, in conjunction with the results obtained from the tested piles, to determine the length of all untested piles. The Engineer will determine the number of blows required to drive the last foot of penetration, in order that the pile will be accepted.

On account of the irregularity of the material in which these piers are built, bidders are requested to examine carefully the borings, as made by the County. In general all piling must penetrate the stiff blue clay which occurs at varying depths.

The price bid per linear foot of concrete piling shall include all materials and labor necessary to drive the same in place. Payment will be made only on piles left in place.

The price bid for each pile tested shall include all labor and material necessary to install a balanced load of at least 70 tons on one concrete pile.

SHEET PILES

By F. H. Avery

Sheet piling differs from any other type of piling in that it is rarely used to furnish vertical support, but is used as a retaining wall. This use is either permanent, as in docks and piers, or temporary, as in cofferdams.

Wood, steel, or reinforced concrete may be used for sheet piling —the latter, of course, in permanent work only.

88. Wood Sheet Piling.—The simplest form of sheet piling consists of a single row of planks driven with their edges touching. Properly braced this will support a bank of earth but will not keep out water (see Fig. 44*a*). Sometimes lighter pieces, called battens, are driven on the outside over the joints (see Fig. 44*b*). Tongue and groove sticks, 3 in. or 4 × 6 in., are sometimes used for small cofferdams (see Fig. 44*c*). A double row of planks of the same thickness may be driven with joints broken and is called shiplap sheet piling (see Fig. 44*d*). This type of piling is often used for the outer wall of puddle cofferdams.

Before the introduction of steel sheet piling, piles were sometimes made of timbers for deep cofferdams by forming rectangular or dovetailed tongues and grooves by nailing wooden strips on the edges of the timbers (see Fig. 44*e*). These timber sheet piles sometimes had the grooves planed out of both edges of the

timbers and a hardwood spline driven in to make the joint (see Fig. 44*f*).

The most common form of timber sheet piling is that known as Wakefield, originally patented, but on which the patents have expired. It is made of three thicknesses of plank with the middle plank offset to form a tongue and groove (see Figs. 44*g* and 45).

Wakefield piling is usually made of 2, 3, or 4-in. pine or fir, surfaced four sides. The piles are generally made at the site,

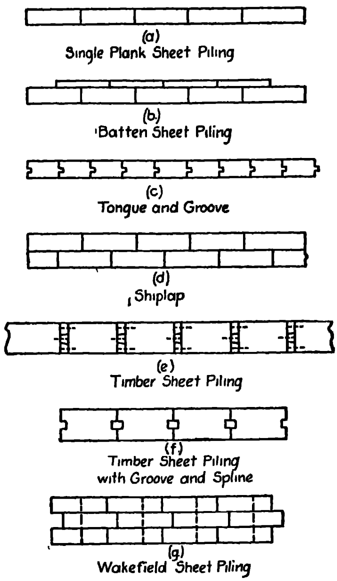


FIG. 44.—Types of timber sheet piling.

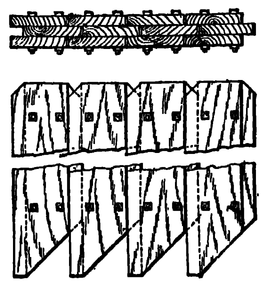


FIG. 45.—Wakefield sheet piling.

using either wire nails or boat spikes. The planking is nailed with the center plank forming a tongue and groove by nailing a portion to the weather as follows:

2-in. plank makes tongue 2½ to 3 in.
3-in. plank makes tongue 3½ to 4 in.
4-in. plank makes tongue not over 4 in.

This gives piling about 5, 8, and 11 in. thick and the length of nails should be 6, 9 and 12 in. respectively. This will allow ½ in. for clinching.

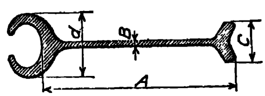
One-half the nails should be driven from one side of the pile and then the pile should be turned over and the remaining half driven. The nails should be driven on 1-ft. centers for 3 to 6 ft. at each end of the pile and 3 to 4 ft. apart in the center portion of the pile. Thus, in a short pile 12 to 15 ft. long, the first three

nails from either end are on 1-ft. centers while in a 27 or 30-ft. pile the end 6 ft. would have nails driven on 1-ft. centers. The pile is completed by making a 45-deg. cut on the lower end beginning about 3 in. from the tongue and by cutting off the upper corners at 45 deg. for about 3 or 4 in. to prevent brooming.

89. Steel Sheet Piling.—During the present century, steel sheet piling has come into general use in the United States, particularly for deep cofferdams and for single wall cofferdams which have to be watertight.

Steel sheet piling is made by a number of manufacturers, each making special claims for their own product, such as interlocking features, general shape of the cross-section, and ability to form offsets and turn corners.

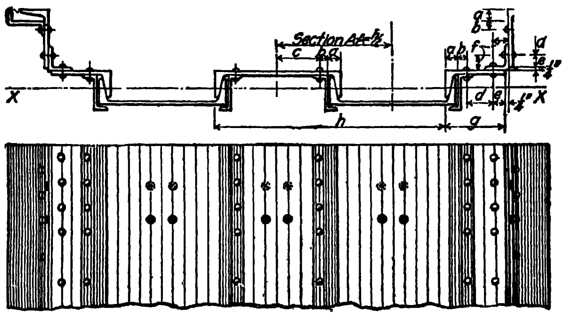
U. S. Steel Sheet Piling



Weight per linear foot (pounds)	*A* (inches)	*B* (inches)	*C* (inches)	*D* (inches)
43	12½	½	2⅝	4$\frac{1}{16}$
38	12½	⅜	2½	3$\frac{15}{16}$
16	9	¼	1⅜	2$\frac{9}{16}$

Friestedt Interlocking Channel Bar Piling

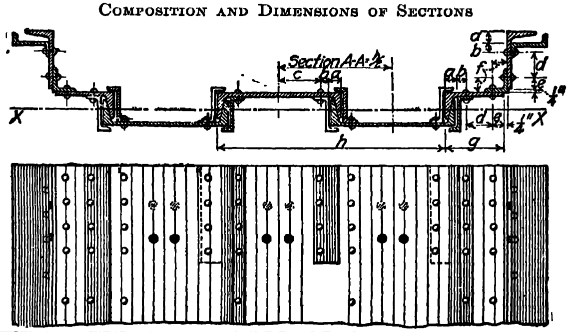
Composition and Dimensions of Sections



Description	Channels		Zees		a (in.)	b (in.)	c (in.)	d (in.)	e (in.)	f (in.)	g (in.)	h/2 (in.)
	(in.)	(lb.) per ft.	(in.)	(lb.) per ft.								
12″ × 33 lb.	12	20.5	3⅜ × ⅜	8.6	1⅜	1¼	3⅜	2¼	1⅛	1⅜	6	10⅞
12″ × 38 lb.	12	25.0	3⅝ × ⅜	8.6	1⅜	1¼	3⅜	2¼	1⅛	1⅜	6	10⅞
15″ × 38 lb.	15	33.0	4⅛ × ⅜	9.2	1 11/16	1 5/16	4½	3	1½	1¾	7½	13½
15″ × 44 lb.	15	40.0	4⅛ × ⅜	9.2	1 11/16	1 5/16	4½	3	1½	1¾	7½	13½

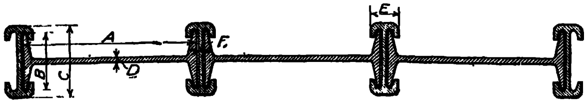
Symmetrical Interlock Channel Bar Piling

Composition and Dimensions of Sections



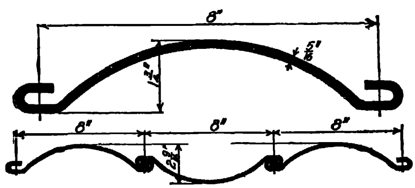
Description	Channels		Zees		a (in.)	b (in.)	c (in.)	d (in.)	e (in.)	f (in.)	g (in.)	h/2 (in.)
	(in.)	(lb.) per ft.	(in.)	(lb.) per ft.								
10″ × 28 lb.	10	15.0	3⅛ × ¼	4.8	1 1/16	15/16	3	2	1	1¼	5	9
10″ × 34 lb.	10	20.0	3⅛ × ¼	4.8	1 1/16	15/16	3	2	1	1¼	6	9
12″ × 34 lb.	12	20.5	3⅝ × ⅜	8.6	1⅜	1¼	3⅜	2¼	1⅛	1⅜	6	10⅞
12″ × 39 lb.	12	25.0	3⅝ × ⅜	8.6	1⅜	1¼	3⅜	2¼	1⅛	1⅜	5	10⅞
15″ × 39 lb.	15	33.0	4⅛ × ⅜	9.2	1 11/16	1 5/16	4½	3	1½	1¾	7½	13½
15″ × 45 lb.	15	40.0	4⅛ × ⅜	9.2	1 11/16	1 5/16	4½	3	1½	1¾	7½	13½

JONES AND LAUGHLIN STEEL SHEET PILING



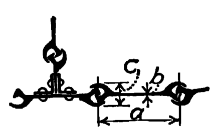
Size (inches)	Weight per sq. ft. (pounds)	*A*	*B*	*C*	*D*	*E*	*F*
12 × 5	35.00	12	3.94	5	0.34	1.97	0.21
12 × 5	36.25	12	3.97	5	0.37	1.97	0.21
15 × 6	37.20	15	4.75	6	0.37	2.12	0.23
15 × 6	39.75	15	4.81	6	0.44	2.12	0.23
15 × 6	42.25	15	4.87	6	0.50	2.12	0.23

LACKAWANNA PLATE SHEET PILING



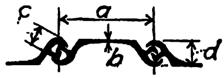
Width: 8 in. between joint centers.
Thickness of metal: $\frac{3}{16}$ in.
Thickness of wall: $2\frac{9}{16}$ in.
Weight per sq. ft. of wall: 11.50 lb.
Weight per linear foot of piling section: 7.66 lb.

LACKAWANNA STEEL SHEET PILING—STRAIGHT WEB TYPE



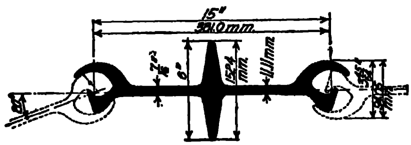
a (inches)	*b* (inches)	*c* (inches)	Weight per linear foot (pounds)	Weight per sq. ft. (pounds)
7	$\frac{1}{4}$	$1\frac{33}{64}$	12.54	21.5
$12\frac{3}{4}$	$\frac{3}{8}$	$3\frac{45}{64}$	31.18	35
$12\frac{3}{4}$	$\frac{1}{2}$	$3\frac{45}{64}$	42.5	40

LACKAWANNA STEEL SHEET PILING—ARCHED WEB TYPE



a (inches)	b (inches)	d (inches)	Weight per linear foot (pounds)	Weight per sq. ft. (pounds)
14	3/8	3 11/16	40.83	35.0
15	7/16	4 1/8	58.12	46.5

LACKAWANNA CENTER FLANGE STEEL SHEET PILING



This type (center flange) is designed for construction requiring high tensile and compressive strength in the pile section, together with a fairly high transverse strength. The center flange, as rolled on this section, acts as a stiffener to the web, increases the modulus of the section, and furnishes a means for attaching transverse ties, braces, etc., needed in special work, and tie rods used in binding on protective concrete facing in permanent protected construction.[1]

The Wemlinger Steel Piling produces steel sheet piling consisting of heavy corrugated steel sheets with interlocking devices (see Fig. 46).

A special form of steel piling was developed by the Great Lakes Dredge and Dock Company for use in the cofferdam for the dock wall at the Chicago Union Station (see Fig. 47).

FIG. 46.—Wemlinger sheet piling.

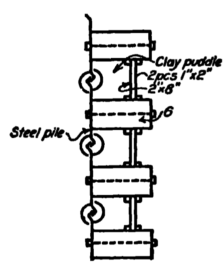


FIG. 47.—Combination sheet piling.

Various other sections using structural channels and beams and special rolled sections have been developed by other steel companies.

[1] From Lackawanna Catalogue.

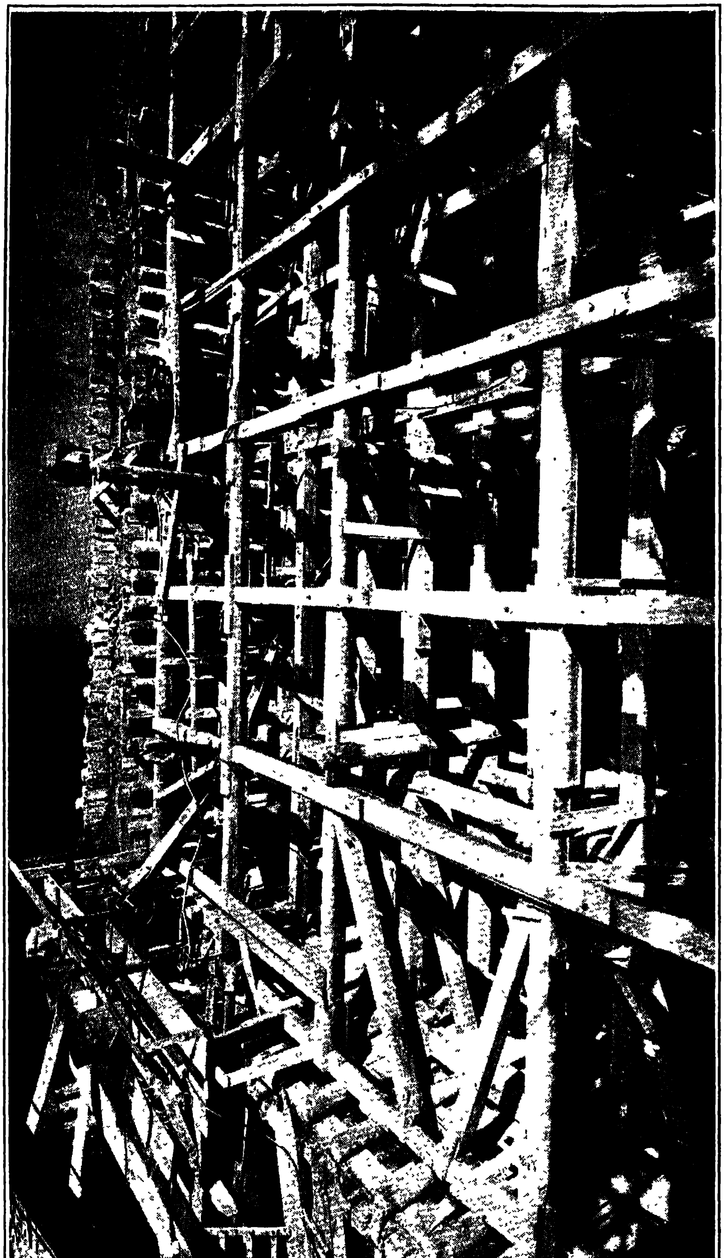


FIG. 48.—A well braced cofferdam with puddle wall using wooden sheet piling for both walls.

Corners for steel pile cofferdams are made by riveting structural angles to the webs of the two halves of a standard sheet pile. Sometimes the web is bent to an angle of 90 deg. It is usually

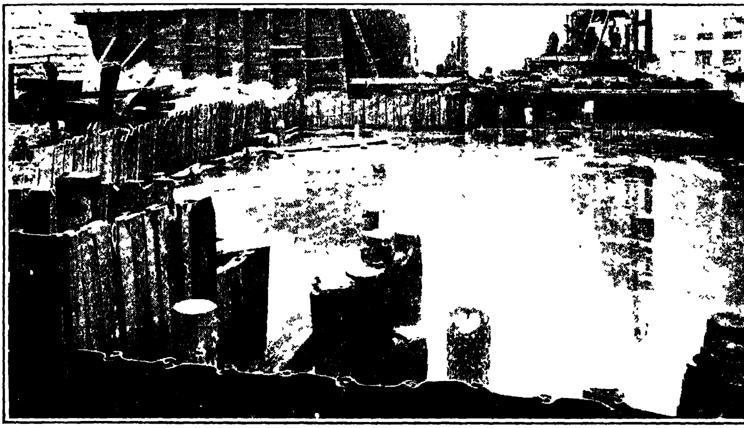


Fig. 49.—Cofferdam showing Lackawanna arch web piling for front and side walls—Wakefield piling for back wall.

possible to complete a closed dam by the use of standard sections only by using care in the driving of the last few sections, and

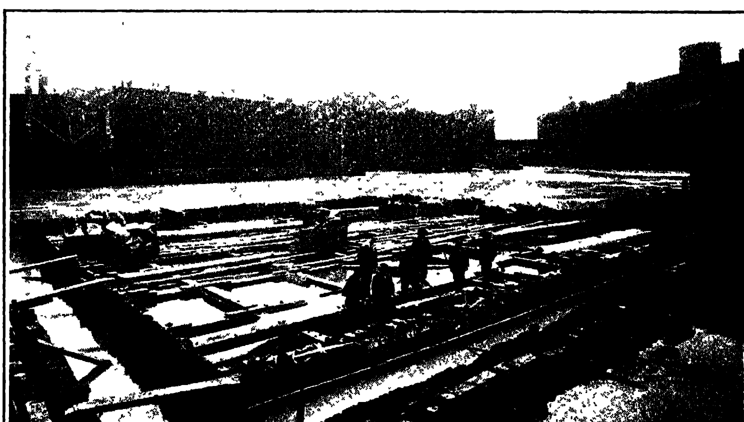


Fig. 50.—Puddle wall cofferdam showing combination of wooden and steel sheet piling.

sometimes assembling several sections and driving them together. When necessary, a closing piece can be made by splitting a stand-

ard pile and riveting the pieces together to give the desired width. Tapered closing pieces are made in the same way. Sometimes closing pieces are bolted together with slotted holes to give the necessary flexibility.

Steel sheet piling has been used extensively since its first development for cofferdams in the Chicago harbor. It is quite common to find wooden shiplap, or Wakefield piling, used for the outer walls of puddle dams and Wakefield or Lackawanna arched web steel piling used for the inner walls. Steel sheet piling is also common for single wall cofferdams. Wakefield was used successfully for the single wall cofferdam of the C. M. and St. P. Ry. bridge over the North Branch of the Chicago River, but this is quite unusual. Wakefield is often used up to 30 ft. in length; above that length steel piling is almost always used. The common length for steel sheet piling around Chicago is 40 ft.

Wakefield piling is sometimes used over more than once; steel piling, however, has often been used eight times over on cofferdam work. The manufacturers of U. S. Steel Sheet Piling claim that under favorable conditions some of their piling has been driven, pulled and re-driven over fifty times.

90. Driving Sheet Piling.—Wakefield piling is still driven to quite a large extent by drop hammers as it stands up well under impact. They are pressed tight into the groove by means of a snub line, the loop end of which is dropped over a piece of piling left about 1 ft. high and several feet back of the driving face; the other end is kicked down 3 or 4 ft. on the pile and then taken to a niggerhead and a strain taken on it. The line can be greased to prevent cutting. After the pile is driven to place it should be toe nailed to the cap and then a face nail driven close to the back or grooved side to prevent the pile from springing until the cap and liners are nailed.

Steel sheet piling is driven almost entirely with steam hammers. Driving caps of cast, or structural steel with wood cushion blocks are used when the driving is hard. The makers of the various kinds of steel piling have caps designed to fit their piles. Light steam hammers have been developed for driving small sheet piles; they can be handled by one man and are so arranged that the weight of the man is added to that of the hammer. Wooden sheet piling on small jobs is often driven by hand, using heavy wooden mauls.

91. Straightening Sheet Piling.—Steel sheet piling can be straightened after pulling. Slight bends are often taken out by piling the crooked pieces with straight pieces on top of them. Straightening bends and kinks often takes considerable work as shown by the following time record: 1 pile-driver crew straightened 22 piles in 5 hr., then spent 3 hr. getting ready, handling piling, etc.; the next day they straightened 32 pieces in 8 hr.

92. Watertightness.—Leaks in steel piling dams are best closed by pouring cinders into the water on the outside of the dam close to the leaking joint. Various devices have been tried to close up the interlock space and seal it, but they are apt to give trouble in driving and pulling and this simple method is the one most successful.

93. Durability.—Steel piling is claimed to have a life of 100 yr. in clear fresh water. Both the United States and Lackawanna companies recommend their piling for use in permanent work with a concrete protection completely enclosing the steel and mechanically bonded to it.

94. Reinforced Concrete Sheet Piling.—Reinforced concrete sheet piling has been patented in various forms and used to some extent in this country. It can probably best be designed for each individual case. Special care should be taken to provide sufficient strength in all directions to withstand the strains of handling and driving as well as its permanent load.

SECTION 4

SPREAD FOOTINGS

FOOTING AREAS

By Albert M. Wolf

1. Relative Allowable Pressure on Soil.—The bearing power of various soils has been discussed in a general way in Section 1, but in actual foundation design for a building it will be necessary to modify or discount these values in accordance with the height and character of loading of the building. That is, for the same soil conditions a lesser bearing value should be used for a high building than for a low (one- or two-story) building since any inequality in the loading of the building, or in the bearing value of the soil, might result in a serious overload on the soil in that section and possible failure due to unequal settling of the high structure, while the low one under the same circumstances might be little harmed. Then again, the relative bearing value of the same soil should be taken as less for a large multi-storied warehouse designed for heavy floor loads, than for a low (three- or four-story) building with comparatively light floor loads. Briefly, this means that any table of bearing values for various soils should be used with a great deal of discretion and modified to correspond with what experience has taught to be safe values for any certain district.

If the soil on which the footings are to rest is likely to flow under load, or be disturbed by other foundation work adjacent, or by seepage or drainage into sewers, special precautions should be taken to retain the soil. This is especially necessary where the foundation bed is of wet sand which might be pumped out in keeping the water out of excavations. Lines of sheet piling of concrete, steel, or wood, driven down to a depth below which any subsequent excavation is unlikely to go will usually give the desired results.

For the footings of buildings without basements the footings need only be carried down below the maximum frost line. This rule holds good for footings on solid rock as well as in earth since

much damage can be done by the freezing of water which may find its way into fissures in the rock under footings. After getting below the frost line, in general it will not be economical to excavate deeper unless a soil of greater bearing capacity can be found at such depth as will make the saving in concrete and steel in the footing, due to the lesser area, greater than the extra cost of excavation. However, in compressible soils, such as various clays, and in wet sands, the footings should be carried down below the line of possible danger of disturbance or lateral displacement of the soil by adjacent building operations, or to such depth that the weight of the soil above will prevent heaving at the periphery of footings under load.

The bearing power of the soil under many buildings has been improved by drainage of the foundation bed by means of lines of drain tile laid adjacent to the exterior or wall footings and slightly below the bottom of these footings. In this way any ground or surface water, which may find its way to the level of the bottom of the footings and tend to lower the bearing value of the soil by the attendant softening thereof, is conducted away from the foundations. Where such drains are laid in sand, the joints of the tile should be carefully wrapped with burlap to prevent the entrance of the sand, for the movement of the sand might undermine the footings.

Sometimes heavy layers of sand or gravel have been placed in bottoms of excavations in poor soils in order to improve the bearing capacity, but this method is of extremely doubtful value if the undersoil is soft, for when put under load the tendency is for the added material to squeeze into the natural soil and so cause settlement. The better method of increasing the allowable bearing on compressible foundation beds such as clays, is to drive short piles as close to each other as possible over the foundation area, thereby compressing the soil and raising its bearing power.

2. Proportioning Footings.—The aim in all footing design where the foundation bed is at all compressible is to have the unit bearing pressure as nearly uniform under various conditions of loading as is possible, in order that the settlement, if any, may be uniform. The present day methods of monolithic reinforced concrete construction greatly reduce the possibilities of unequal settlement since the strength of the connecting units—columns, slabs, beams, or girders, or all combined—act as a stiff frame work,

transferring some of the load to adjacent column footings, where one footing tends to settle unequally, thus relieving the situation.

To have the bearing pressure under all footings uniform or very nearly so, is impossible of attainment under all conditions of loading because of the fact that the interior columns carry a greater percentage of live load than the exterior columns. The problem, therefore, resolves itself into approximating equality of soil pressure by making the footing areas proportional to loads which include only a part of the live load—say 50 per cent or less, or none of the live load—depending on the character of the occupancy. This results in relatively larger footing areas for exterior columns (for a given bearing value) as compared with the interior than would be the case if the full live load were considered in proportioning the areas of footings.

The loads to be considered on building foundations are: (1) the dead load of the building, (2) the live or movable loads to which the floors may be subjected, and (3) the wind loads. The latter loads are in general neglected on buildings having a width as great or greater than the height, or where the side walls are protected by other buildings. For very narrow and high buildings it is essential that the wind loads be considered, for in buildings only two or three bays wide the loads on the leeward footings may be considerably increased by wind loads and unless they are proportioned accordingly, unequal settlement is very likely to occur.

The dead load, or the weight of the structure itself including walls and partitions, can be readily computed. The maximum allowable load can also be readily computed, but it is evident that a building very seldom carries the maximum allowable live load over the entire area of each floor at the same time. Aisle spaces, unloaded areas, and partially loaded areas generally considerably reduce the actual live loads on the various floors, and it would therefore be wrong to proportion the columns and footings for the full allowable live load for which the floors may be designed.

The usual practice in building design, therefore, is to design the floor slabs for the full allowable live load per square foot, the girders for 85 per cent of this allowable, or assumed live load, and a further reduction is made on the amount of live load carried by the building columns. The reduction of live load for columns varies with different city ordinances, the idea in general being to design the columns as nearly as possible for the probable actual

loads they will receive owing to the conditions mentioned in the preceding paragraph.

The Chicago Building Code fixes the amount of live load to be used in computing the live load carried by the columns as follows: For the roof, full live load; for the first floor below the roof, 85 per cent of the live load; and for each succeeding floor below, a further reduction of 5 per cent until a reduction of 50 per cent is reached, after which no further reduction is allowable. Not considering the roof load—as this is usually only 25 lb. per sq. ft.—the above reduction formula gives the live load carried to the footing as 67.5 per cent of the total allowable live load (based on the assumption that the allowable live load on all floors is the same).

Other column load reduction formulas are used, the more common being that recommended by the National Board of Fire Underwriters, namely:

> In buildings more than five stories in height, the following reductions are permissible: For columns supporting roof and top floor, no reduction; for columns supporting each succeeding floor, a reduction of 5 per cent of the total of live load per floor may be made, but the total reduction shall not exceed 50 per cent.
>
> No reduction of live load on columns shall be permitted in buildings where the assumed floor load is more than 120 lb. per sq. ft. and is likely to be permanent in character, as in warehouses, printing houses, machine shops, etc.
>
> For structures carrying machinery, such as cranes, conveyors, printing presses, etc., at least 25 per cent shall be added to the stresses from live loads to provide for impact and vibrations.

These two latter requirements of the Underwriters' Code seem just a bit severe and increase the cost of a building considerably by the greater size of columns required.

The lower story column live load, as arrived at by the above reduction formulas, is the load for which the maximum stresses in the footing should be computed. For interior column footings this load should be used together with the dead load to find the area of footing required, using the maximum allowable bearing pressure on the soil in question.

After having found the footing area required, divide the sum of the dead load and 50 per cent of the lower story column live load (as reduced) by said area, and a new bearing value will be found. Now take the exterior column dead load plus 50 per cent of the lower story column live load (as reduced), and divide by the new

bearing value found above. The result will be the area for exterior footings. In computing stresses in the exterior footings, however, the lower story column live load as reduced (not 50 per cent of said live load), plus the dead load, should be used.

Illustrative Problem.—Determine column footing areas for a three-story and basement building with floor loads of 200 lb. per sq. ft., roof load of 25 lb. per sq. ft. and panels 20 × 20 ft. Column loads are to be reduced in accordance with the Chicago Building Code. Maximum allowable soil pressure is 4000 lb. per sq. ft., neglecting weight of footing.

Interior column load:

Dead load = 193,000 lb.
Live load = 202,000 lb.

Total = 395,000 lb.
Dead load plus 50 per cent of live load = 294,000 lb.

Exterior column load:

Dead load = 143,000 lb.
Live load = 106,000 lb.

Total = 249,000 lb.
Dead load plus 50 per cent of live load = 196,000 lb.

Interior column footing area required

$$= \frac{395,000}{4000} = 99 \text{ sq. ft.}$$

Make footing 10 ft. square = 100 sq. ft.

Pressure on soil for dead load plus 50 per cent live load

$$= \frac{294,000}{100} = 2940 \text{ lb. per sq. ft.}$$

$$\text{Area of exterior footing required} = \frac{196,000}{2940} = 66.6 \text{ sq. ft.}$$

Make footing 8 ft. 2 in. square.

In computing stresses in the footing, the pressure for full live and dead load must be taken, this equals $\frac{249,000}{66} = 3740$ lb. per sq. ft. From this it will be noted that while under total live and dead load the pressure under the interior footings is 4000 lb. per sq. ft., under the exterior footings it will be only 3740 lb., which indicates that the design is such as to preclude any marked difference in settlement of the exterior and interior footings. For dead load only the pressure under the exterior footing equals 2166 lb. per sq. ft. and under interior column footing equals 1930 lb. per sq. ft.

In cases where the assumed live load is very small—say 40 lb. per sq. ft.—it may be well to proportion the footings for dead load only, but in general it will be better practice to include a certain amount of the lower story column live load, say 30 per cent for buildings with light loads, and 50 per cent or more for buildings with heavy live loads of a more or less permanent character.

3. Eccentricity in Footings.—Wherever possible, footings should be so designed as to have the center of pressure coincide with the center of the base of the footing. This, however, is not always possible of attainment since under some conditions a combination of wind or earth pressure (or both) with the dead and live loads may be such as to make the line of resultant pressure depart from the vertical and intersect the base of footing at some point beyond the center. In such a case the footing is said to be loaded eccentrically and the pressure on the soil is not uniform, but varies from a minimum at one side to a maximum at the other.

If the line of resultant pressure lies within the middle third of the footing (in which case e, the eccentricity, is less than $\frac{1}{6}l$, where l equals the dimension of the footing base in the plane under consideration), the pressure on the soil will vary from a minimum at the edge farthest from the point of intersection of the resultant with the base to a maximum at the opposite side. The variation in the pressure for this condition is shown in Fig. 1, Case (1).

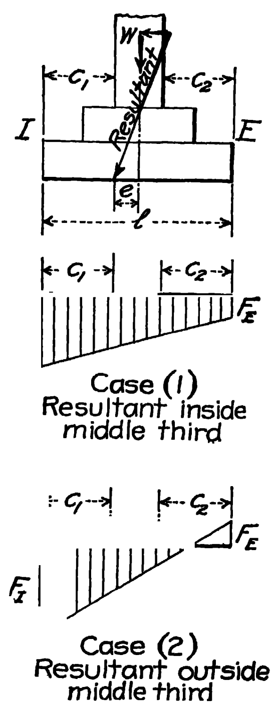


FIG. 1.

The actual intensities of pressure at the interior and exterior edges of the footing, marked I and E, respectively, are given by the formulas:

$$F_I = \frac{W}{l}\left(1 + \frac{6e}{l}\right)$$

$$F_E = \frac{W}{l}\left(1 - \frac{6e}{l}\right)$$

in which the footing is assumed to be square or rectangular, and W equals the total vertical load on the footing, or the total load per lin. ft. on a wall footing. To obtain the unit bearing pressure, the value of F_I or F_E should be divided by the width of footing (1 ft. in the case of the wall footing).

If the resultant of pressure intersects the base outside the middle third, the eccentricity will be greater than $\frac{1}{6}l$ and the pressure on the base will vary from a condition of no pressure

or an uplift on the side farthest away to a maximum at the side nearest to the point of intersection of the resultant with the base. This condition is shown in Case (2), Fig. 1, where

$$F_I = \frac{W}{3e} = \frac{2W}{l'}$$

where l' is the effective dimension of the footing actually under load and equals six times the eccentricity e.

The condition of pressure shown in Case (2), however, should never be allowed in good design especially in buildings, and when investigation indicates a footing to be so stressed it should be redesigned.

CONCRETE FOOTINGS

4. Wall Footings.—A building of the wall bearing type (that is, where no exterior columns are used—the floor slabs resting directly on the exterior brick or concrete walls) will usually require the footing for the exterior wall to be reinforced as a balanced cantilever projecting beyond each face of the wall an equal distance. In residence work or in larger buildings where the bearing power of the soil is high, the wall footings will have a small projection and are usually constructed of plain concrete, the usual practice being to make the thickness of each footing course twice the projection of the course beyond the course above.

In designing a reinforced concrete wall footing the bending moment at any section of the footing at a distance x from the end (Fig. 2) will be

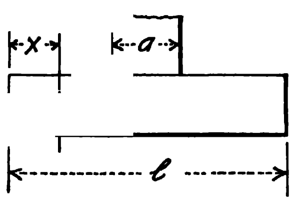


FIG. 2.

$$M = \frac{1}{2} wx^2$$

where w = the uniform bearing pressure per lin. ft. of footing for a given width of section. Now if l is the extreme width of footing and a is the thickness of wall, the bending moment at the face of the wall will be

$$M = \frac{1}{8} w (l - a)^2$$

For a section at the center of wall the bending moment will be a maximum and equal to

$$M = \frac{1}{8} w (l^2 - la)$$

However, since the resisting moment will be much greater at this point than at any in the footing projection, owing to the greater depth of wall, the critical section will be at the face of wall. That this is actually the case is borne out by Talbot's tests on reinforced concrete footings.[1] These tests indicate that the maximum tensile stresses developed at the face of wall are somewhat less than the calculated stresses even when the wall and footing were not cast at the same time.

In designing reinforced concrete cantilever footings for walls, special consideration should be given to the ascertaining of bond stress and diagonal tension. In computing the maximum bond stress on bars, Talbot's tests show that the total external shear at the face of the wall should be used in the formula for unit bond stress. In computing the shear for diagonal tension, however, it should be taken on a line at a distance away from the wall equal to the effective depth of the footing. In view of this fact diagonal tension is quite likely to govern in the design of footings composed of a number of steps or with a sloped top, since the depth is generally less at a distance (d) from the face of the wall than at the wall. As a general rule it will be found better practice, from the standpoints of design and construction, to keep down the amount of diagonal tension reinforcement by making the footing courses relatively thick. In any event the reinforcement for diagonal tension should be bent to template and proper supports provided to ensure its proper placement and location in the finished work.

5. Types of Column Footings.—Column footings may be divided into four principal types depending on the number of columns the footing supports, namely: (1) Isolated or single footings supporting one column; (2) combined footings to carry two or more columns; (3) cantilever footings, usually supporting one exterior and one interior column; (4) continuous footings, supporting a line of columns, or all the columns of a building on continuous strips of footings at right angles to (and integral with) each other, or on a mat covering the entire lot area.

6. Single or Isolated Column Footings.

6*a*. Plain Concrete Footings.—Plain concrete footings are the natural outgrowth of the now almost obsolete stone masonry footings in which each of the courses forming the footing

[1] Univ. of Ill. Eng. Exp. Sta. *Bull.* 67.

act as cantilever beams projecting beyond the next course above and are uniformly loaded. Since no reinforcement is used, the design must be such that all projections will have a thickness sufficient to keep the tensile stress in the concrete well within the allowable under the maximum condition of loading.

The University of Illinois tests on plain footings gave results showing considerable variation which did not permit a method of determining the effective width of resisting section to be established or to obtain a formula for resisting moment. Based upon the full section of the footing, the moduli of rupture obtained were considerably less than the moduli of rupture of control beams made with the same concrete.

In view of these facts it seems that the better practice to follow in the design of such footings is to so proportion them as to eliminate all bending stresses.

In a reinforced concrete footing the load is transmitted to the soil over its entire area by virtue of the deflection or deformation of the footing under load, while in a plain concrete footing on a hard soil or rock the load tends to distribute only over such area as lies within the base of a pyramid or cone formed by the lines of stress from the base of the column to the bottom of the footing. The general practice is, therefore, based on the assumption that the load is carried through the concrete at an angle of 30 deg. with the vertical. If all the projections lie outside of a line drawn at 30 deg. with the vertical from the edge of column to the bottom of the footing, no bending stresses will exist. The simplest form of a plain concrete footing would therefore seem to be a pyramid or cone, but owing to the difficulty of holding the forms on footings of this shape, stepped or coursed footings are used with all projections lying outside of the above mentioned line of stress. To design a plain concrete footing on this basis, first find the area of footing required, and from one-half the width of bottom course subtract one-half the column size, and divide the result by the tan. 30 deg. This will give the required height of footing. Then divide the footing into as many vertical steps as desired keeping the projections entirely outside of a 30-deg. line with the vertical from edge of footing to edge of column. If this method is followed, the safe punching shear value of 120 lb. per sq. in. will never be exceeded in the footing.

In stepping off plain concrete footings the steps should be at least 12 in. high and preferably more. The area of the top course

should be such as will allow the maximum bearing value on the footing concrete directly under the column base.

For footings on rock or on soil capable of sustaining relatively high unit loads, plain concrete footings should be used rather than reinforced concrete since owing to the unyielding character of the foundation the reinforced concrete footing could not act as designed.

Where excavation must be carried to a considerable depth below the ground floor line, it will be often found more economical to use plain concrete footings since no saving can be made in excavation, which generally makes for economy in reinforced footings, and then also the footing concrete will usually be cheaper than the extra length of reinforced concrete column required if reinforced footings without plinth blocks are used.

6b. Advantages of Reinforced Concrete Footings.—Except in certain cases as mentioned above where plain concrete footings can be used to advantage, reinforced concrete footings will in normal times be found the most economical, since a saving in excavation, material, and weight of footing itself can be made.

A study made a few years ago as to the relative cost of various shapes of reinforced concrete footings developed the conclusions that footings with sloping tops are more expensive to build, single course footings next, and a decreasing range of cost as more courses, for a given depth, were used in the footings.

6c. Summary of Professor Talbot's Tests.—The fact that the tests made under the direction of Professor Talbot at the University of Illinois were the first made on column footings, makes the phenomena of the tests and data of their action of especial interest to engineers and designers and it is therefore deemed advisable to quote in full the summary of conclusions drawn. Much information as to weaknesses in footings to be guarded against and as to methods of calculation of bending and resisting moments for square footings is given, and this information is of special value in design of footings of any character if properly used.

(1) A square column footing under load may be expected to take a bowl-shaped form. In slabs subject to bending in two directions, the stress in a fiber can not differ from that in an adjoining fiber at the same level without setting up longitudinal shear; and as there is considerable resistance to variation from equality of stress in adjoining fibers, it may be expected that in stiff thick pieces (as are footings of ordinary design, where the thickness is large in comparison with the length of the projection) the deformations and

consequent stresses will be distributed over the width of a cross-section and that considerable stress will be developed even in the fibers at the edge of the footing.

(2) For footings having projections of ordinary dimensions, the critical section for the bending moment for one direction (which in two-way reinforced concrete footings is to be resisted by one set of bars) may be taken to be at a vertical section passing through the face of the pier. In calculating this moment, all the upward load on the rectangle lying between a face of the pier and the edge of the footing is considered to act at a center of pressure located at a point half-way out from the pier, and half of the upward load on the two corner squares is considered to act at a center of pressure located at a point six-tenths of the width of the projection from the given section. By equating this bending moment and the resisting moment which is available at the given section, the maximum tensile stress in the concrete or in the reinforcing bars may be calculated.

(3) As is usually the case when plain concrete is used in flexure, the unreinforced footings show considerable variation in results. The variations were such as not to permit a method of determining the effective width of resisting section to be established or to obtain a formula for resisting moment. Based upon the full section of the footing, the moduli of rupture obtained were considerably less than the moduli of rupture of control beams made with the same concrete.

(4) In reinforced concrete column footings, resistance to non-uniformity of stress in adjoining bars will be given by bond and by longitudinal shear in the concrete, and the amount of variation from uniformity of stress in the various bars will depend upon the spacing of the bars as well as upon the relative dimensions of the footing. With two-way reinforcement evenly spaced over the footing, it seems that the tensile stress is approximately the same in bars lying within a space somewhat greater than the width of the pier and that there is also considerable stress in the bars which lie near the edges of the footing. For intermediate bars stresses intermediate in amount will be developed. For footings having two-way reinforcement spaced uniformly over the footing, the method proposed for determining the maximum tensile stress in the reinforcing bars, is to use in the calculation of resisting moment at a section at the face of the pier the area of all the bars which lie within a width of footing equal to the width of pier plus twice the thickness of footing, plus half the remaining distance on each side to the edge of the footing. This method gives results in keeping with the results of tests. When the spacing through the middle of the width of the footing is closer, or even when the bars are concentrated in the middle portion, the same method may be applied without serious error. Enough reinforcement should be placed in the outer portion to prevent the concentration of tension cracks in the concrete and to provide for other distribution stress.

(5) The method proposed for calculating maximum bond stress in column footings having two-way reinforcement evenly spaced, or spaced as noted in the preceding paragraph, is to use the ordinary bond stress formula, and to consider the circumference of all the bars which were used in the calculation of tensile stress, and to take for the external shear that amount of upward pressure or load which was used in the calculation of the bending moment at the given section.

An important conclusion of the tests is that bond resistance is one of the most important features of strength of column footings, and probably much more important than has been appreciated by the average designer. The calculations of bond stress in footings of ordinary dimensions where large reinforcing bars are used show that the bond stress may be the governing element of strength. The tests show that in multiple-way reinforcement a special phenomenon affects the problem and that lower bond resistance may be found in footings than in beams. Longitudinal cracks form under and along the reinforcing bar due to the stretch in the reinforcing bars which extend in another direction, and these cracks act to reduce the bond resistance. The development of these cracks along the reinforcing bars must be expected in service under high tensile stresses, and low working bond stresses should be selected. An advantage will be found in placing under the bars a thickness of concrete of two inches, or better three inches, for footings of the size ordinarily used in buildings.

Difficulty may be found in providing the necessary bond resistance, and this points to an advantage in these of bars of small size, even if they must be closely spaced. Generally speaking, bars of ¾-in. size or smaller will be found to serve the purpose of footings of usual dimensions. The use of large bars, because of ease in placing, leads to the construction of footings which are insecure in bond resistance. In the tests the column footings which were reinforced with deformed bars developed high bond resistance. Curving the bar upward and backward at the end increased the bond resistance, but this form is awkward in construction. Reinforcement formed by bending long bars in a series of horizontal loops covering the whole footing gave a footing with high bond resistance.

(6) As a means of measuring resistance to diagonal tension failure, the vertical shearing stress calculated by using the vertical sections formed upon the square which lies at a distance from the face of the pier equal to the depth of the footing was used. This calculation gives values of the shearing stress, for the footings which failed by diagonal tension, which agree fairly closely with the values which have been obtained in tests of simple beams. The formula used in this calculation is

$$v = \frac{V}{bjd}$$

where V is the total vertical shear at this section taken to be equal to the upward pressure of the area of the footing outside of the section considered, b is the total distance around the four sides of the section, and jd is the distance from the center of reinforcing bars to the center of the compressive stresses. This stress is somewhat larger than the average vertical shear over the section which is sometimes used. The working stress now frequently specified for this purpose in the design of beams, 40 lb. per sq. in., for 1 : 2 : 4 concrete, may be applied to the design of footings.

The punching shear may be calculated for the vertical sections which inclose the pier footing, although it may be expected that shear failure may not be produced exactly on this section. The value now generally accepted for punching shear, 120 lb. per sq. in. for 1 : 2 : 4 concrete, may be used for the working stress in this case.

(7) No failures of concrete in compression were observed, and none would be expected with the low percentages of reinforcement used. The compressive stresses in the pier of the footing were in some cases very high and in a few instances the pier failed and was replaced by a cube of concrete. In frequent cases there were signs of distress near the intersection of pier and footing where there is an abrupt change in direction of surfaces and where the combined stresses are very high.

(8) In stepped footings, the abrupt change in the value of the arm of the resisting moment at the point where the depth of footing changes may be expected to produce a correspondingly abrupt increase of stress in the reinforcing bars. Where the step is large in comparison with the projection, the bond stress must become abnormally large. It is evident that the distribution of bond stress is quite different from that in a footing of uniform thickness. The sloped footing also gives a distribution of stress which is different from that in a footing of uniform thickness. However, for footings of uniform thickness the bond stress is a maximum at the section of the face of the pier; in a sloped footing the bond stress at the section at the face of the pier would be less accordingly than in a footing of uniform thickness, and a moderate slope may be found to distribute the bond stress more uniformly throughout the length of the bar. This is not of advantage if the full embedment of the bar is effective in resisting any pull due to bond.

(9) The use of short bars placed with their ends staggered increases the tendency to fail by bond and cannot be considered as acceptable practice in footings of ordinary proportions. In footings in which the projection is short in comparison with the depth the objection is very great.

(10) Footings having reinforcement placed in the direction of the diagonals as well as parallel to the sides (four-way reinforcement) gave good tests. The significance of the results is so obscured by the variety of manner of failure (bond, diagonal tension, and perhaps tension) and by variations in the quality of the concrete, that a comparison with two-way reinforcement on the basis of loads carried would not be of value. This type of distribution of reinforcement should be included in further tests. Measurements of deformation in the bars are needed to determine the division of stress among the four sets of bars.

6*d*. Design of Isolated Footings. *Maximum Bending Moment.*—Based on the foregoing conclusions of Talbot's tests a formula for determining bending moments in column footings has been derived which is as follows:

$$M = \left(\frac{1}{2}ac^2 + 0.6c^3\right)w$$

where a is one dimension of the column or pier, c the offset of the footing, and w the unit bearing pressure.

It will be of interest in view of Talbot's tests with its resultant formula to compare the values of moment given thereby with others used quite extensively before the publication of the tests. Assume load to be carried by the footing 500,000 lb., allowable

bearing on soil 5000 lb. per sq. ft., size of footing 10 ft. square, $a = 2$ ft., $c = 4$ ft. (see Fig. 3).

Method 1.—Load assumed to be carried by portion of footing beyond face of pier for full width of footing. Load on corners is therefore taken in account twice.

$$M = (w)(c)(l)\left(\frac{c}{2}\right) = wl\frac{c^2}{2}$$

$$= (5{,}000)(10)\frac{(4)^2}{2} = 400{,}000 \text{ ft.-lb.}$$

Method 2.—One-fourth of the total load considered as applied at the center of gravity of a triangle formed by the diagonals from center to corners of footings.

$$M = (w)\left(\frac{l}{2}\right)\left(\frac{l}{2}\right)\left(\frac{l}{3}\right) = \frac{wl^3}{12}$$

$$= (5{,}000)\frac{(10)^3}{12} = 416{,}600 \text{ ft.-lb.}$$

Method 3.—Full load considered on a cantilever of width a and length c, with one half load on corners, inasmuch as they are taken into account twice.

$$M = \frac{wc^2}{4}(l + a)$$

$$= \frac{(5{,}000)(4)^2}{4}(10 + 2) = 240{,}000 \text{ ft.-lb.}$$

Method 4.—First find depth of footing required for punching shear, then take the load on a cantilever beyond a line formed by the intersection of a plane, having a slope of 1 (horizontal) to 2 (vertical) from the

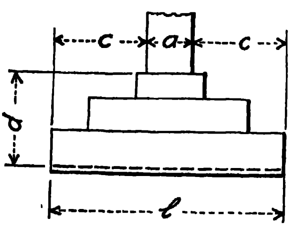


FIG. 3.

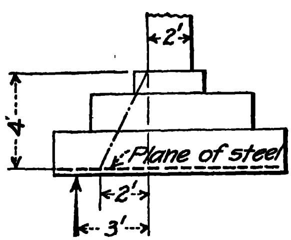


FIG. 4.

face of column, with the base. Consider load applied at center of cantilever and the moment arm equal to the distance from face of pier or column to this center. The assumption in this method is that part of the load is transferred directly through the footing to soil within the four planes sloping as above described from the four column faces and produces no moment. The depth to steel

$$d = \frac{500{,}000}{(96)(120)} = 43.5 \text{ in. net}$$

Use 48-in. total depth to properly protect reinforcement (see Fig. 4). With a 4-ft. depth, the direct stress plane intersects the base 2 ft. out from the face of the column, leaving a 2-ft. cantilever. The total load carried

on this portion = (2)(10)(5,000) = 100,000 lb. The moment arm = 3 ft.

$$M = (100,000)(3) = 300,000 \text{ ft.-lb.}$$

Method 5.—Talbot's Formula:

$$M = \left(\frac{1}{2} ac^2 + 0.6c^3\right) w$$

$$= 5,000(1 + 4^2 + 0.6 \times 4^3) = 277,000 \text{ ft.-lb.}$$

From the above it will be seen that Methods **4** and **5** give results very nearly the same. The use of Talbot's formula is recommended in preference to any of the others owing to its agreement with test results.

From the foregoing comparisons it will be seen that while Methods **1** and **2** give results considerably higher than the Talbot formula, Methods **3** and **4** give results just a trifle under and over respectively of those given by the Talbot formula and show that designers using these formulas were working very close to what tests showed to be the action of footings.

Depth for Punching Shear.—In order that a footing shall not fail due to the column punching through it, the punching shear on an area equal to the perimeter of the column times the depth to the steel reinforcement shall not exceed the allowable value of 120 lb. per sq. in. for 1:2:4 concrete. The load producing punching shear shall be taken as the column load less the unit bearing pressure times the area of column, since this latter load does not produce shear on the area in question. The load producing punching shear can be found by multiplying the column load by the ratio

$$\frac{\text{footing area minus column area}}{\text{footing area}}.$$

Width of Footing to Use in Flexure Computations for Two-Way Reinforcement.—Professor Talbot's tests indicate that, as a working basis applicable when the spacing of the bars is uniform or does not vary from this, the resisting moment of the footing in each of the two directions may be based upon the amount of steel in a width of beam equal to the width of pier plus twice the depth of the footing to the reinforcement plus one-half the remainder of the width of footing and that the use of this amount of steel will determine the maximum steel stress. Expressed as a formula the equivalent beam width then is

$$b = a + 2d + \frac{1}{2}(l - a - 2d)$$

In no case, however, should the beam width be taken greater than the width of footing.

Bond Stresses.—The bond stresses may be based upon the shear at the section at the face of the pier. For this the external vertical shear will be the amount of load used in determining the critical bending moment. At the face of the pier this shear is

$$V = \frac{1}{4}(l^2 - a^2)w = (ac + c^2)w$$

The expression for bond stress is

$$u = \frac{V}{\Sigma 0 jd}$$

where Σ is the number of reinforcing bars included within the equivalent width of beam as used in calculating the maximum tensile stress (see Conclusion **5** of Talbot's tests in Art. **6***c* regarding sizes of bars to be used in order to provide sufficient bond resistance).

Diagonal Tension.—As a means of measuring resistance to diagonal tension failure, the vertical shearing stress calculated by using the vertical sections formed upon the square which lies at a distance from the face of the pier equal to the depth of the footing was used. The external vertical shear V may be considered to be that part of the load on the footing outside of the sections considered. The following formula expresses the amount of the vertical shear by this assumption:

$$V = [l^2 - (a + 2d)^2]w$$

The expression for the critical vertical shearing stress becomes:

$$v = \frac{V}{4(a + 2d)jd}$$

The working stress for shear as a measure of diagonal tension shall be taken as 40 lb. per sq. in. for 1:2:4 concrete.

7. Combined Column Footings.—Where the exterior building columns are so located with respect to the property lines that sufficient bearing area cannot be obtained for a symmetrical isolated footing, a combined footing can be used. In such a footing, the exterior column in question is carried on a common footing with the adjacent interior column, the footing being of such size and shape as to give the required bearing area and to make the center of gravity of the loads coincide with the center of the upward reaction. This is essential in order that if any settlement should occur it will be as nearly uniform as the character of the soil will allow and also to avoid dangerous transverse

stresses in columns. At corners of buildings it often becomes necessary due to the above mentioned restrictions, to place four columns on a combined footing, which may be solid or with a portion of the center omitted if the bearing value of the soil is relatively high.

Various shapes of combined footings can be used depending upon the relation of the column loads, the allowable projection of the footing beyond the respective column center lines and whether or not relatively high shearing stresses are allowed.

From the standpoint of economy the rectangular shaped footing with a greater thickness under columns (to take care of punching and diagonal shear) is the best. In such a footing fewer different lengths of bars are required, the transverse bending stresses are reduced to a minimum and also the amount of diagonal tension and direct moment reinforcement. The two latter reductions are possible because of the greater thickness of slab at columns which in turn cuts down the span of slab, in the same manner as the flaring heads in flat-slab construction.

If a footing of uniform thickness is used, the span producing moment is greater and the shearing stresses will either be high or the depth for moment excessive. Such footings should, therefore, not be used except for small footings or where it is necessary to keep the footings relatively shallow because of soil conditions.

From a construction standpoint the simpler the reinforcement and shape of footing the cheaper will be the cost. The construction engineer detests nothing more than an irregular shaped footing loaded with stirrups and bent bars of different lengths, for when working below ground level many difficulties are met with which mitigate against obtaining a first class job if the layout of reinforcement is complicated.

In general no limitation is placed on the allowable projection of footing beyond the interior column and therefore, an illustrative example of a rectangular combined footing is given in Art. **10** since it represents the best and most common practice.

8. Cantilever Footings.—The type of footing described in Art. **7** namely, the combined footing in which two columns are carried on the same footing so proportioned as to have the center of gravity of the loads coincide with the center of gravity of the upward reaction—is sometimes confused with, or even called a cantilever footing, which, of course, is not correct. A cantilever

footing is a construction connecting the footings of an interior and of an exterior column, the latter because of obstructions or local conditions being so placed as to have its center of gravity eccentric, with the center of gravity of the column. This necessitates a strap or beam connection with the interior footing which transfers the uplift, caused by cantilevering the exterior column beyond the center of the footing area supporting it, to the interior column footing. This construction is used where it is necessary to avoid encroachment on adjacent property or streets.

In this type of footing the uplift created at the footing for the interior column is found by multiplying the exterior column load by the eccentricity of the footing and dividing by the distance between the center of gravity of the exterior footing and the interior column. The strap connecting the two footings must be sufficiently strong to resist the bending moment caused by the eccentricity and the shear at the interior column. In construction the strap beam should be built so as not to bear on the foundation bed which would complicate the action of the footing. This requirement can be met by excavating below the line of the bottom of the strap and building forms above. The space underneath should be boxed off so as to prevent filling in under the strap beam form (see Fig. 15, p. 239).

9. Continuous Footings.—Continuous footings may be divided into two main classes depending upon whether or not they are continuous between one or more lines of columns in one line and at right angles thereto. Where they are continuous between columns in one row only, as is often the case for wall columns where it is necessary to keep the projection beyond the building to a minimum owing to building code or property line restrictions they are usually called continuous footings; while if they cover the entire lot or are composed of several strips at right angles to each other built monolithic and supporting all of the columns they have generally been called raft footings. This latter name has been applied since such footings should be used only where the bearing power of the soil is very low and the function of the footing is literally to "float" the building on a raft covering the entire, or a large percentage of the ground area occupied by the building. They may also be used where the soil conditions require pile foundations, and the building loads are so heavy as to require a large number of piles which make it necessary to cover practically the entire building area with a "raft."

When soil conditions seem to warrant the use of continuous footings, the engineer should, by very careful study and tentative design, determine whether or not this type will be more economical and satisfactory than pile foundations, the next logical choice. Where investigation of the soil conditions shows a soil strata of greater carrying capacity, underlying the one in which the footings would be placed under ordinary circumstances, at a depth not to exceed 25 or 30 ft. below the latter, and there is any danger of the upper strata being disturbed or settling materially due to adjacent building operations, it will usually be found more economical and a more stable foundation secured by using concrete or wood pile foundations or circular piers carried down to the solid stratum in wood-sheeted wells.

Continuous footings are best adapted to clay soils of low bearing power where the tendency to unequal settlement due to unequal loading of various parts of the building can be counteracted by tying the entire structure together as a large box, thus making adjoining portions of the footing aid the overloaded one in carrying the load. The action of such a footing under load is analogous to what tests show happens in a flat slab floor under unequal loading.

In cases where it is necessary to load the entire foundation area, the footing slab can be designed as a flat slab floor. In designing, however, it will be well to guard against undue bending stresses in the exterior columns produced by eccentricity of loading if the slab or mat does not project beyond the building lines sufficiently to balance the load. It should be remembered, however, that, where the exterior columns act as buttresses or supports for the basement walls, the pressure against these will relieve or counteract a certain amount of the bending induced by the eccentricity of the footing load.

Instead of using the ordinary sloping column head, so common in flat slab construction, as a base for the column resting on the footing slab it will be found more economical to use a square or oblong pedestal consisting of one or more courses of sufficient size and depth to meet the requirements of accepted flat slab design. For cases where the column spacing is over 20 ft. it may be found more economical to use a paneled slab for the mat —that is, one with greater thickness for the portions of the slab containing the main reinforcing running direct between columns.

A considerable saving in concrete may thus be effected by such footing slabs which should be designed by the same methods as used for "paneled ceiling" construction.

An example of a continuous footing supporting all columns of the building on several strips of reinforced concrete slabs at

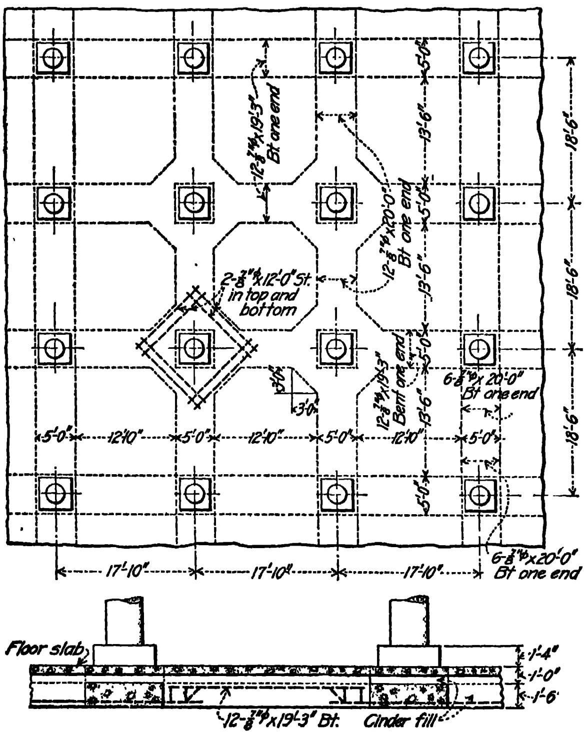


Fig. 5.

right angles to each other is shown in Fig. 5. Note the method of obtaining additional bearing area for columns supporting the sprinkler tank in addition to the regular building loads.

An example of a raft foundation composed for beams or girders at right angles to each other with a relatively thin slab underneath to distribute the load over the entire area is shown in Fig. 6.[1] This raft was designed as a T-beam structure and while a saving in the design was effected in this respect, it requires an extra fill and another slab for the basement floor over the footing slab proper. A raft of flat slab construction eliminates the

[1] From "Engineering News," Aug. 26, 1909, p. 212, by W. A. Hoyt.

necessity of another slab for the basement floor provided the column bases required for shear and moment considerations are not so large as to take up a considerable added area in the basement.

10. Illustrative Problems in the Design of Isolated, Combined, and Cantilever Footings of Reinforced Concrete.—*Design Data.*—

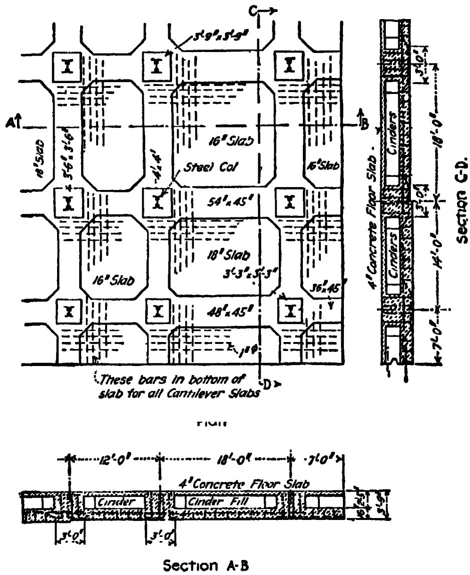


Fig. 6.

Design the necessary interior and exterior column footings for a three story and basement building with interior panels 20 ft. by 20 ft., center to center of columns, and with exterior exposed columns placed so that the distance from center of first row of interior columns to outside face of exterior columns is 20 ft. (Fig. 7). Floor loads are 200 lb. per sq. ft. Building is to occupy a lot 80 ft. wide by 200 ft. long with street on one side and one end, alley at the other end, and an inside lot line on the other side. On the long street side, footings can project only 18 in. beyond the lot line by city ordinance because of future proposed subways. On the other street there are no restrictions. On the inside property line, outer faces of columns are to be on the line. Maximum allowable soil pressure is 4000 lb. per sq. ft., neglecting weight of footing. Sizes of columns are as follows:

interior columns, 24-in. diameter; exterior columns, 24 × 36-in.; corner columns, 18 × 36-in. The column loads are listed below,

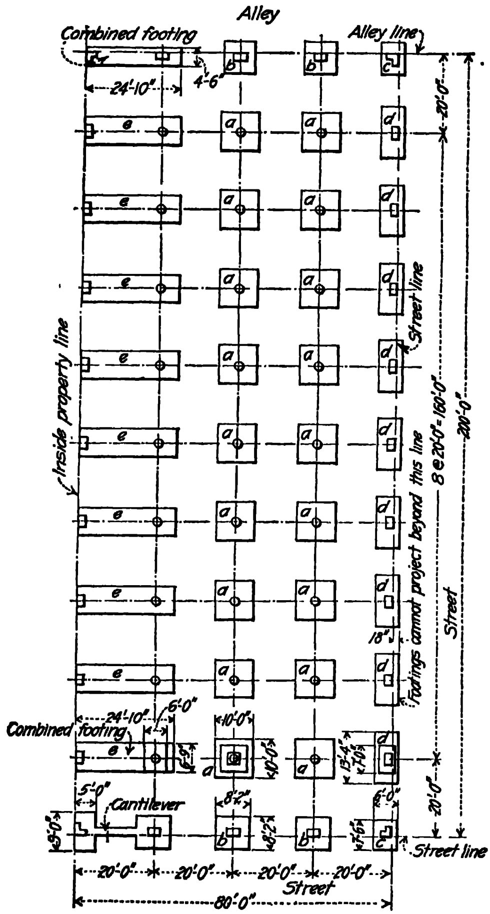


Fig. 7.

the live load being reduced in accordance with the Chicago code.

Column	Dead load	Live load	D.L. + L.L.	D.L. + 50% L.L.
Typical interior................	193,000	202,000	395,000	294,000
Typical ext. column...........	143,000	106,000	249,000	196,000
Corner column................	127,000	54,000	158,000	131,000

Areas of Footings.—Interior column footing area $= \frac{395,000}{4,000} =$ 99 sq. ft.

Use a 10×10-ft. footing = 100 sq. ft.

With a footing of this size the pressure on soil for dead load plus 50 per cent of the column live load is $\frac{294,000}{100} = 2,940$ lb. per sq. ft.

Exterior column footings should be proportioned for dead load plus 50 per cent of the live load, hence the bearing value of 2,940 lb. should be used in determining the area required, which equals $\frac{196,000}{2,940} = 66.6$ sq. ft.

Use a footing 8 ft. 2 in. square on alley and short street side. On the longer street side the limitation of 18-in. projection beyond building line and column thickness of 24 in. restricts width of footing to 5 ft. [2 ft. + (2×18 in.)]. Therefore the length of footings must be 13 ft. 4 in. For full live load plus dead load the pressure under exterior footings will be $\frac{249,000}{66.6} = 3,740$ lb. per sq. ft. This pressure must be used in design of reinforcement for footing.

The corner column footings require an area equal to $\frac{131,000}{2,940} =$ 44.5 sq. ft. Owing to the fact that the limitation of projection of footing on the longer street side is 18 in. beyond building line and the center line of column is 18 in. in from building line (exterior columns have 36-in. exposed faces) the corner footings on street and alley corners can be 6 ft. wide and the length 7 ft. 6 in. giving an area of 45 sq. ft. For full live load and dead load the pressure equals $\frac{158,000}{45} = 3,500$ lb. per sq. ft. This pressure should be used in analyzing stresses in corner footings.

The footings along the inside property line will have to be combined with those in the next row away from the property line because of the eccentricity of column as regards the possible footing area. For a typical exterior and interior column the combined area required equals 166.6 sq. ft. The center of gravity of the combined footing must coincide with the center of gravity of loads. The center of gravity of loads equals $\frac{(294,000)(19)}{490,000} = 11.4$ ft. from the center of exterior column (taking moments about

center of exterior column). Hence the footing must be 2(11.4 + 1) = 24.8 ft. or 24 ft. 9½ in. long. Use 24 ft. 10 in. in computations. The width equals $\frac{(100 + 66.6)}{24.8} = 6.72$ ft. Use width of 6 ft. 9 in. The dimensions of footing are shown in Fig. 14.

At the street and alley ends adjacent to the property line, a corner and an exterior column must be combined and an area of 66.6 + 44.5 = 111.1 sq. ft. is required. The center of gravity of loads equals $\frac{(196{,}000)(19)}{327{,}000} = 11.4$ ft. from center of corner column. It so happens for the loads given that the length of this footing will be the same as the one just proportioned carrying an interior and an exterior column, but this, of course, is not always the case. Using a length of 2(11.4 + 1) = 24.8 ft., the width required is $\frac{111.1}{24.8} = 4.5$ ft. The size of the various footings for the building as above determined are shown in Fig. 7.

Detailed Design of Typical Interior Footing.—As determined above, the typical interior footings are to be 10 ft. square. The load producing punching shear equals

$$\frac{100-3.1}{100} = (395{,}000) = 383{,}000 \text{ lb.}$$

The minimum depth for punching shear is found by dividing the load producing punching shear by the periphery of the column times the allowable unit punching shear or 120 lb.

$$\text{Depth} = \frac{383{,}000}{(75.4)(120)} = 42.4 \text{ in.}$$

Use a 42½ in. depth to steel, with 3½ in. below center of steel, making total depth 46 in.

The shear as a measure of the diagonal tension is measured at a distance from the face of the column equal to the depth of the footing to steel, or 42½ in. in this case.

The area between the square formed by lines 42½ in. from face of column and the edge of footing produces shear on the vertical planes through the line *EFGH* (Fig. 8).

$$\text{Total hear on } EFGH = \frac{100 - (9.08)^2}{100}(395{,}000) = 69{,}500 \text{ lb.}$$

Using a unit shear value of 40 lb. per sq. in., the depth necessary at the plane in question may be found and the footing designed accordingly. Since $v = \frac{V}{bjd}$ and $d = \frac{V}{vbj}$, and also since j may be assumed as 0.875 with sufficient accuracy,

$$d = \frac{69{,}500}{(40)(4)(109)(0.875)} = 4.56 \text{ in.}$$

The area of the top of footing on which the column bears should be at least twice the area of the column, in which case a bearing of 600 lb. per sq. in. is allowable under Chicago Ordinance and 700 lb. per sq. in. under Joint Committee Rules. The balance of the load in the column must be transmitted to the footing by stub bars embedded for one-half their length in the footing (see also Art. 12). For the footing in question the top area should be at least 6.28 sq. ft. For practical purposes the projection beyond column face should be at least 6 in. so that the column forms can be rested thereon without difficulty. This means that the top of footing will be 3 ft. square. If sloping sides are used, they should be sloped from the 3-ft. square line to a line giving at least 1-ft. total depth of footing at edge. This will meet all shear requirements, as above computation shows. If a stepped footing is used, a good practical rule to follow is to keep the steps outside of a line drawn from the center of the column to the edge of the base in the plane of the reinforcement (Fig. 9). The outline of a sloped footing is also shown in Fig. 9.

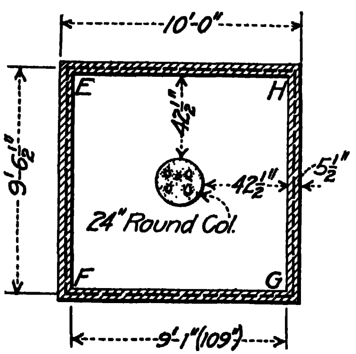


Fig. 8.

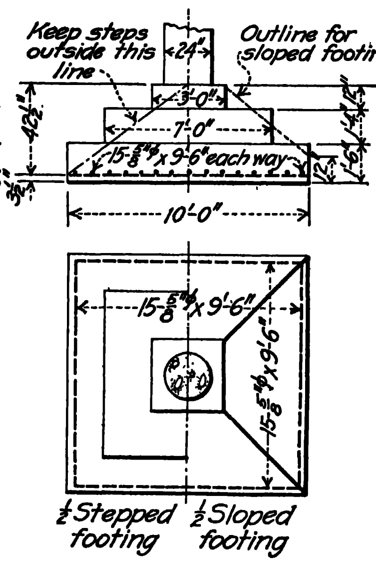


Fig. 9.

Testing for shear at the face of the next to bottom step for the stepped footing we have

$$v = \frac{\frac{100 - (7)^2}{100}(395{,}000)}{(4)(84)(0.875)(18)} = 38 \text{ lb. per sq. in.}$$

This indicates that the outline of footing is such as will not require the use of stirrups to take care of shear in the concrete. Except where local or special conditions demand it, footings should be designed so as to avoid the necessity of using stirrups.

The bending moment on each set of rods is

$$M = (\tfrac{1}{2}\, ac^2 + 0.6\, c^3)w$$

Since $a = 2$ ft. and $c = 4$ ft.

$M = [(1)(4)^2 + (0.6)(4)^3]\ (4{,}000)\ (12) = 2{,}659{,}200$ in.-lb. The area of steel required, (A_s), assuming an allowable steel stress (f_s) of 16,000 per sq. in.

$$A_s = \frac{M}{f_s jd} = \frac{2{,}659{,}200}{(16{,}000)(0.875)(42.5)} = 4.45 \text{ sq. in.}$$

The effective width of footing equals the diameter of column plus twice the thickness of footing plus one-half the remaining distance on each side to the edge of the footing, or

$$24 + [(2)(42\tfrac{1}{2})] + 5\tfrac{1}{2} = 114.5 \text{ in.} = 9 \text{ ft. } 6\tfrac{1}{2} \text{ in.}$$

We will assume, inasmuch as no bars would be placed closer than 3 in. from edge of footing, that if placed in a band 9 ft. 6 in. wide all bars will be effective in resisting bending. For the area of steel required in each band, fifteen ⅝-in. round bars will be ample.

The bond stress on each set of bars.

$$u = \frac{V}{\Sigma ojd} = \frac{(383{,}000)(0.25)}{(1.96)(15)(0.875)(42.5)} = 87.5 \text{ lb. per sq. in.}$$

If deformed round bars are used with an allowable bond stress of 100 lb. per sq. in., the number and size of bars above selected will be satisfactory while if plain round bars are used, a larger number of smaller bars will be required to keep the bond stress within the allowable 80 lb. per sq. in.

Design of Rectangular Exterior Column Footing.—The square footings of exterior columns on alley and short street side will not be designed in detail here, but the rectangular footings along the long street side will be in order to indicate the procedure with footings of this type.

These footings are 5 ft. wide by 13 ft. 4 in. long (Fig. 10).

The load producing punching shear equals

$$\frac{66.6-6}{66.6}(249{,}000) = 226{,}600 \text{ lb.}$$

$$\text{Depth required} = \frac{226{,}600}{(120)(120)} = 15.8 \text{ in.}$$

Use 16-in. depth to steel and 20 in. total depth. Inasmuch as

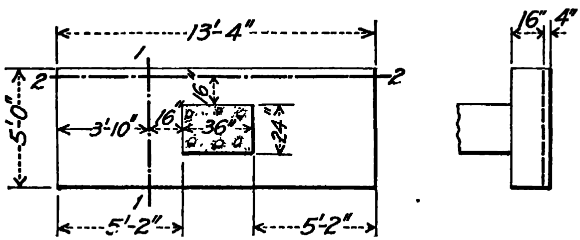


FIG. 10.

the footing is relatively shallow, a single course with 20-in. total thickness will be used.

The shear as a measure of diagonal tension across the width of footing at a line *1-1*, 16 in. out from end of column (Fig. 10) equals (5) (3,740) (3.83) = 71,600 lb.

$$v = \frac{71{,}600}{(60)(0.875)(16)} = 85 \text{ lb. per sq. in.}$$

The shear at line *2-2* will be very small owing to narrowness of footing, and therefore need not be considered.

The above indicates that while a depth of 16 in. will be sufficient for punching shear, the shear as a measure of the diagonal

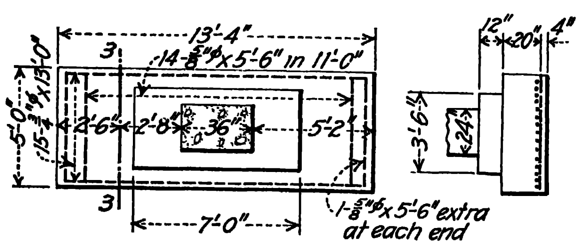


FIG. 11.

tension will be high at the line *1-1*, and, in order to avoid the use of stirrups, the depth will be increased. This can be done by adding a block of concrete 3 ft. 6 in. wide by 7 ft. long, and 12 in. thick, and increasing the depth of bottom course to 20 in. to center

of steel. The shear as a measure of diagonal tension at line *3-3* in Fig. 11 will then equal (5) (3,740) (2.5) = 46,750 lb.

$$v = \frac{46{,}750}{(60)(0.875)(20)} = 44 \text{ lb. (which is close enough for practical purposes).}$$

The bending moments for rectangular footings are given by the following formulas:

Where the length is not greater than 50 per cent more than the breadth (Fig. 12),

$$M_{(1-1)} = (\tfrac{1}{2}ac^2 + 0.6c^2b)w$$
$$M_{(2-2)} = (\tfrac{1}{2}ab^2 + 0.6cb^2)w$$

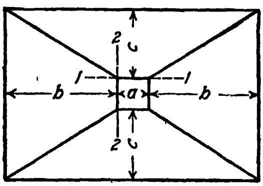


Fig. 12.

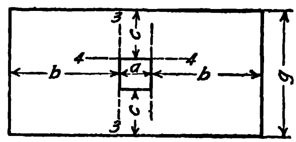


Fig. 13.

Where the length exceeds the breadth by more than 50 per cent, as in Fig. 13, the moments are:

$$M_{(3-3)} = \left(\frac{wb^2}{2}\right)g$$
$$M_{(4-4)} = \left(\frac{ac^2}{2} + \frac{bc^2}{2}\right)w$$

The footing in question comes under the case shown in Fig. 13 and the moment about the line corresponding to *3-3* equals

$$M_{(3-3)} = \frac{(3{,}740)(5.17)^2}{2}(5)(12) = 2{,}998{,}000 \text{ in.-lb.}$$

$$M_{(4-4)} = \left[\frac{(2)(3)^2}{2} + \frac{(5.17)(3)^2}{2}\right](3{,}740)(12) = 1{,}851{,}000 \text{ in.-lb.}$$

$$A_{s(3-3)} = \frac{2{,}998{,}000}{(16{,}000)(0.875)(32)} = 6.67 \text{ sq. in.}$$

Use fifteen ¾-in. round bars.

$$\text{Bond stress} = u = \frac{(25.85)(3{,}740)}{(2.35)(15)(0.875)(32)} = 98 \text{ lb. per sq. in.}$$

Deformed bars must be used.

$$A_{s(4-4)} = 4.15 \text{ sq. in.}$$

Use fourteen ⅝-in. round bars.

$$A_s = 4.30 \text{ sq. in.}$$

Bond stress = 55 lb. per sq. in. Effective width of footing = 10 ft. 10 in.

The ⅝-in. bars should, therefore, be spaced in a width of 11 ft. and one additional bar placed beyond them in the remaining width of footing at each end (Fig. 11).

Combined Footings—Rectangular.—The size of the rectangular combined footing for one interior and one exterior column was found to be 24 ft. 10 in. long by 6 ft. 9 in. wide (Fig. 14).

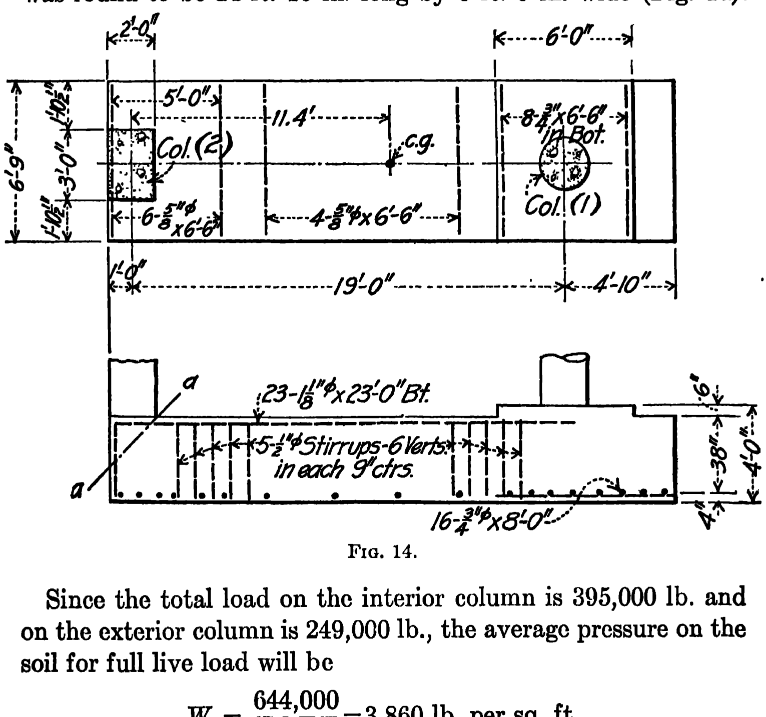


FIG. 14.

Since the total load on the interior column is 395,000 lb. and on the exterior column is 249,000 lb., the average pressure on the soil for full live load will be

$$W = \frac{644{,}000}{166.6} = 3{,}860 \text{ lb. per sq. ft.}$$

This pressure will be used in the detail design of footing. In this, as in the other examples, the maximum allowable soil pressure has been assumed at 4000 lb. per sq. ft. and the weight of footing neglected in the computations. In other words, the soil has been assumed to be capable of bearing the additional load due to weight of footing.

The negative moment due to the overhang of footing beyond the center of interior column 1 will reduce the maximum moment in mid portion of the footing and amounts to

$$M_{(1\text{-}1)} = (6.75)(4.83)\left(\frac{4.83}{2}\right)(12)(3{,}860) = 3{,}640{,}000 \text{ in.-lb.}$$

Maximum moment between columns 1 and 2 $= \frac{wl}{8} - \frac{1}{2}M_{(1\text{-}1)} =$

$$(3{,}860)(6.75)(19)^2(^{12}\!/_{8}) - 1{,}820{,}000 = 12{,}276{,}000 \text{ in.-lb.}$$

Depth required for moment

$$d^2 = \frac{M}{kb} = \frac{12{,}276{,}000}{(108)(6.75)(12)} = 1{,}405$$

$$d = 37.6 \text{ in., say } 38 \text{ in.}$$

The depth required for punching shear at interior column

$$d_1 = \frac{395{,}000 - (3.14)(3{,}860)}{(75.4)(120)} = 42.5 \text{ in.}$$

At exterior column (only three faces of column effective in punching shear)

$$d_2 = \frac{249{,}000 - (6)(3{,}860)}{(84)(120)} = 22.5 \text{ in.}$$

Taking into consideration the depths required for moment and punching shear, a depth of 38 in. to center of steel (42 in. total) will be used for the main footing slab and an extra block of concrete (6 in. thick and 6 ft. wide with a length equal to the width of footing) will be used under column 1.

Using the depth of 38 in. the area of steel required in bottom of slab at column 1 equals

$$A_s = \frac{3{,}640{,}000}{(16{,}000)(0.875)(38)} = 6.85 \text{ sq. in.}$$

Use sixteen ¾-in. rounds = 7.06 sq. in.

$$\text{Bond stress } u = \frac{(3{,}860)(4.83 - 1)(6.75)}{(16)(2.35)(0.875)(38)} = 80 \text{ lb. per sq. in.}$$

Plain bars will be satisfactory.

The area of steel required in top of slab between columns

$$A_s = \frac{12{,}276{,}000}{(16{,}000)(0.875)(38)} = 23 \text{ sq. in.}$$

Use twenty-three 1⅛-in. rounds = 23 sq. in. approximately. Bend these bars down into footing slab at end under column 2 to anchor them and to aid in resisting the tendency of the concrete to shear off on a line *a-a*, Fig. 14.

The shear at side of columns 1 and 2 = 221,000 lb.

$$\text{Bond stress in bars} = u = \frac{221{,}000}{(23)(3.53)(0.875)(38)} = 82 \text{ lb. per sq. in.}$$

Plain round bars will do.

The footing will now be investigated for shear. The shear as a measure of diagonal tension is taken at a line at a distance d from face of column, and equals (5.33)(6.75)(3,860) = 139,000 lb.

$$v = \frac{139,000}{(6.75)(12)(0.875)(38)} = 52 \text{ lb. per sq. in.}$$

This means that stirrups will have to be used in the footing.

A_s for stirrups $= \dfrac{139,000}{16,000}(\tfrac{2}{3}) = 5.8$ sq. in. in a 38-in. length of footing.

Five ½-in. round stirrups with 6 verticals each will give 5.85 sq. in. of steel. These will be spaced at 9-in. centers at each column.

The transverse bending in the footing should next be investigated.

At column 1:

$$M = \left(\frac{395,000}{2}\right)\left(\frac{6.75-2}{6.75}\right)\left(\frac{2.37}{2}\right)(12) = 1,975,000 \text{ in.-lb.}$$

$$A_s = \frac{1,975,000}{(16,000)(0.875)(44)} = 3.2 \text{ sq. in.}$$

Use eight ¾-in. round bars in a width of 6 ft., allowable effective width = 2 ft. plus 7 ft. 4 in. or 9 ft. 4 in.; 6 ft. used since this is width of footing of 44-in. depth.

$$u = \frac{53,500}{(8)(2.35)(0.875)(44)} = 74 \text{ lb. per sq. in.}$$

Plain bars will be satisfactory.

At column 2:

$$M = \left(\frac{249,000}{2}\right)\left(\frac{6.75-2}{6.75}\right)\left(\frac{1.87}{2}\right)(12) = 983,600 \text{ in.-lb.}$$

A_s = 1.85 sq. in.

Use six ⅝-in. rounds in a width of 5 ft. (column plus depth of footing = 62 in.).

Place four ⅝-in. rounds, as distributing bars, in the remaining distance between the two bands of transverse reinforcement above computed.

Combined Footing—Trapezoidal.—Using the same spacing of columns and load as in the previous example, but limiting the projection to 1 ft. beyond column 1 a trapezoidal footing will be required. As determined previously, the center of gravity of the column loads is located 11.4 ft. from the center line of column 2. The center of gravity of the footing should coincide with the

center of gravity of the loads. The length of footing is 22 ft. Now the area required for footing is 166.6 sq. ft., and the average width must be such as to give this area.

If C_1 and C_2 be taken as the width of footing at columns 1 and 2, respectively, then

$$\frac{(C_1 + C_2)(22)}{2} = 166.6$$

or

$$C_1 + C_2 = 15.14 \text{ ft.}$$

From the common equation for center of gravity, the distance from end of footing at column 1 to center of gravity equals

$$x_1 = \left(\frac{22}{3}\right)\left(\frac{C_1 + 2C_2}{C_1 + C_2}\right) = 9.6 \text{ ft.}$$

Solving these equations

$$C_1 = 11.14 \text{ ft. and } C_2 = 4 \text{ ft.}$$

For full live load and dead load

$$w = \frac{644{,}000}{166.6} = 3{,}860 \text{ lb.}$$

This pressure should be used in the design of the footing which from this point is similar to the problem just illustrated except

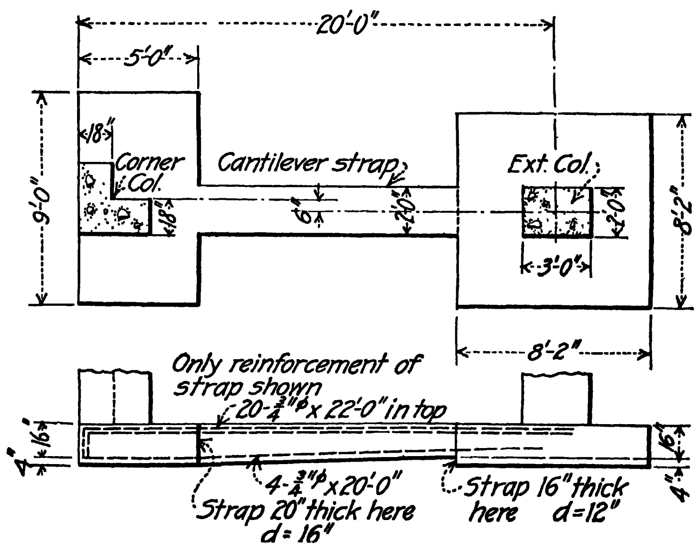


FIG. 15.

that the small negative moment at column 1 may be neglected. The line of maximum moment will be at the line of zero shear.

Cantilever Footing.—As an example of a cantilever footing take the corner footing at the property line and the adjacent exterior column footing and connect them with a strap to resist the uplift moment caused by the eccentricity of the footing slab for corner column (see Fig. 7).

The center of gravity of the corner column is located 18 in. from either face. The area of footing required for corner column is 44.5 sq. ft. In order to reduce the eccentricity, assume a width of 5 ft. for the corner column footing, making the eccentricity 1 ft. (Fig. 15). This will require a length of 9 ft. Owing to the relatively large dimensions of the corner columns required for architectural appearance, a very shallow footing could be used as a computation for punching shear would show (9 in. req'd). However, let us assume the depth to be 16 in. to steel and 20 in. total. For the typical exterior column use a footing 8 ft. 2 in. square as required by previous computation.

By taking moments about the center of gravity of the exterior footing, the uplift for full live and dead load is found to be

$$\frac{(158{,}000)(1)}{17.5} = 9{,}000 \text{ lb.}$$

which load must be applied at the exterior column to balance the moment in the strap beam. The total pressure (exclusive of weight of footing, which is neglected owing to its consideration in fixing the allowable bearing on soil) to be provided for will be 158,000 + 9,000 = 167,000 lb. For the footing size used, this gives a unit bearing of 3,710 lb. per sq. ft.

The footing proper should be designed for this load in a similar manner to the exterior footings along the long street side of building. These computations will not be given here.

The maximum moment in the strap beam equals

$$M = (158{,}000)\ (1)\ (12) = 1{,}896{,}000 \text{ in.-lb.}$$

$$A_s = \frac{1{,}896{,}000}{(16{,}000)\ (0.875)\ (16)} = 8.5 \text{ sq. in.}$$

Use twenty ¾-in. round bars = 8.8 sq. in.

These bars must be placed in the top of strap and the ends at corner column bent into hook form to anchor them, and give adequate shear reinforcement under the column because of its location at the edge of footing.

The section of the strap at the exterior column will be determined by shear. In order to eliminate stirrups in the strap

take shear at 40 lb. per sq. in. and assume the depth of strap to be 12 in. at the column, sloping down to 16-in. depth at the corner column footing. Width required

$$b = \frac{9,000}{(40)(0.875)(12)} = 21.4 \text{ in.}$$

Use 24-in. width for strap and place the 20 ¾-in. round bars in two layers in the top in order to have sufficient space between the bars. Place a nominal amount of reinforcement, say 4 ¾-in. rounds in bottom of strap (see Fig. 15).

11. Illustrative Problems in the Design of Plain Concrete Footings.

11*a*. Footings for Light Walls.—In the design of plain concrete footings for light walls, as for residences and one story brick buildings where the basement or footing walls are made from 8 to 12 in. thick, the projection of the footing beyond the wall should be made one-half the thickness of wall and the depth of footing twice the projection, or the thickness of wall. Thus, for an 8-in. wall, the footing course should be 8 in. thick and have a total width of 16 in., while for a 12-in. wall the footing course should be 12 in. thick and 2 ft. wide.

The footing as above designed should be checked to make sure that the allowable bearing on the soil is not exceeded. This will seldom be found to be the case in buildings of the types mentioned above, except in very poor soils.

11*b*. Footings for Heavy Walls.—Design a footing for a wall of a storage building carrying 21,000 lb. per lin. ft.; allowable bearing on the soil = 3,000 lb. per sq. ft.; wall 24 in. thick.

Assume weight of footing at 3,000 lb. per lin. ft.

Total load per ft. = 21,000 + 3,000 = 24,000 lb.

$$\text{Width of footing} = \frac{24,000}{3,000} = 8 \text{ ft.}$$

This will require three footing courses each 24 in. thick with projections of 12 in. (Fig. 16).

11*c*. Column Footing.—Design a column footing of plain concrete to carry a load of 260,000 lb., the allowable soil pressure being 3,500 lb. per sq. ft.

Column load	260,000
Weight of footing	90,000
Total load	350,000 lb.

$$\text{Area of footing required} = \frac{350{,}000}{3{,}500} \doteq 100 \text{ sq. ft.}$$

Use 10 ft. × 10 ft. for bottom course. Assuming that 1:2:4 concrete is used in the top course of the footing, a stress of 700 lb.

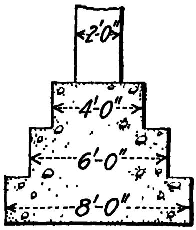


FIG. 16.

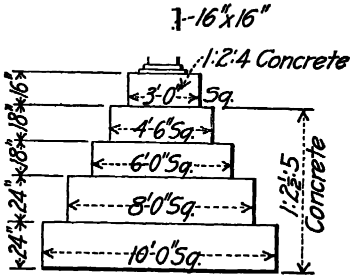


FIG 17.

per sq. in. in bearing can be used. This will require that a cast iron bed plate be used under the column (if of timber) $= \frac{260{,}000}{700}$ $= 371.4$ sq. in. Use a 20 in. square bed plate = 400 sq. in.

The top course will be made 3 ft. square and 16 in. thick, with 8-in. projection. The other courses will be of 1: 2½: 5 concrete of the thickness and projection shown in Fig. 17.

12. Footings Supported on Piles.—Where the bearing value of the soil is so low as to necessitate very large spread footings to properly distribute the load (with the ever attendant danger of movement of the soil under the footing), it will generally be found advisable to use either concrete or wood piles driven down to a firmer stratum, or deep enough to give the required bearing capacity due to skin friction.

Pile footings may be of the same general types as the plain and reinforced concrete footings previously described and illustrated, the difference in design being that instead of assuming a uniform bearing of the soil over the area of the footing, the reaction of each pile is considered as a concentrated load equal to the safe allowable load for the pile.

In figuring the load to be carried on the piles some building ordinances allow a further reduction in live load over the rectangular column live load reduction. Thus the Chicago Ordinance allows the use of the following percentages of the basement column live load to be used in computing the number of piles in footings:

For manufacturing, storage and mercantile buildings, 75 per cent.
For hotels, clubs, hospitals, residences, and apartment buildings, 50 per cent.
For churches, public halls, theaters, and schools, 25 per cent.

The majority of building codes require that the concrete footing capping the piles be carried down 6 in. below the top of the piles and that this concrete be neglected in computing the strength of the footing. Then, also, the spacing of piles is usually limited to 2 ft. 6 in. for wood piles and 3 ft. for concrete piles, except where driven in staggered rows, when the spacing of rows

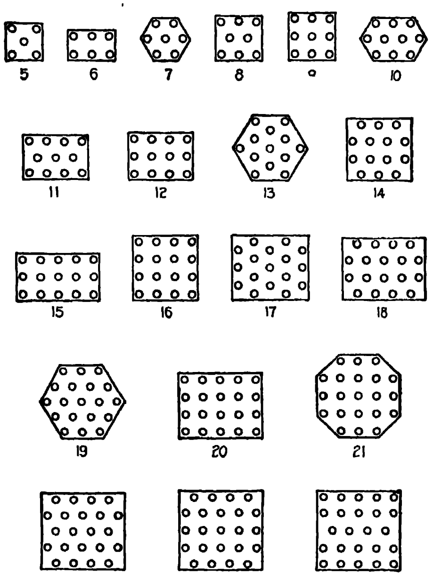


Fig. 18.—Arrangement of piles and shapes of footings.

may be reduced 2 or 3 in. for wood and concrete piles respectively For wood piles, the cut-off line, or top, should be below the natural ground water level to avoid decay. Where the meeting of this requirement means added expense, concrete piles should be used.

The safe load for wood piles in most city building codes is determined by the Wellington formula (see Appendix *B*). This means that while assumptions as to the probable capacity of piles can be made in design, the final design depends upon the driving records for piles and some redesign may be necessary as the work progresses. In Chicago the assumption of a load of 20

tons per pile will usually be found very close to the average allowable load for the piles of any individual footing.

The safe load for concrete piles is usually determined by actual test for each particular job. The general design, however, is made on a basis of an allowable compressive stress in the concrete of 400 lb. per sq. in. in plain concrete piles, with an increase in stresses (as in columns) for piles having steel reinforcement.

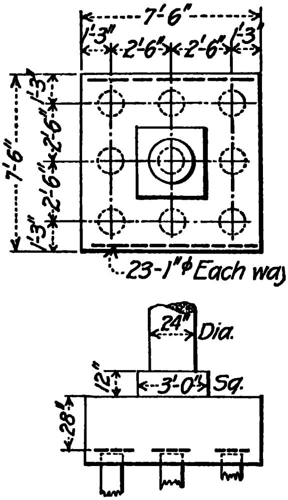


Fig. 19.

The most economical arrangement of piles in footings for various number of piles is shown in Fig. 18, the spacing of piles being as required by the particular code followed.

Illustrative Problem.—To illustrate the general method of design for pile footings, a design will be made for the typical interior column mentioned in Art. **10.** The loads for this column are:

Dead load	= 193,000 lb.
Live load	= 202,000 lb.
Total	= 395,000 lb.
Dead load plus 75 per cent live load	= 344,500 lb.

Assuming 20 tons as the allowable load per pile, the design will be made in accordance with the Chicago Code which allows a reduction of 25 per cent in the live load to be carried by piles in footings for factory, store, or warehouse building. The stresses in the footing proper, however, must be determined for the full lower story column live load.

$$\text{The number of piles required} = \frac{344{,}500}{40{,}000} = 8.6$$

Use 9 piles spaced 2 ft. 6 in. each way with 1 ft. 3 in. from center of outer rows to edge of footing. This will make the bottom course of footing 7 ft. 6 in. square (see Fig. 19).

The depth required for punching shear at the periphery of column

$$d = \frac{395{,}000 - 44{,}000}{(75.4)(120)} = 39 \text{ in.}$$

If the top course is made 3 ft. square, the depth required at its periphery will be

$$d = \frac{395{,}000 - 44{,}000}{(4)(36)(120)} = 20.4 \text{ in.}$$

(In the above, the full load on one pile at center of footing is deducted as not producing punching shear.)

Considering diagonal tension, the depth required for the lower course of the footing, considering two piles producing shear on each side,

$$d = \frac{(2)(44{,}000)}{(7.5)(12)(0.875)(40)} = 28 \text{ in.}$$

Use top course 12 in. thick and bottom course 28 in. effective thickness, making a total depth to center of steel (1 in. above top of piles) 40 in.

The moment at the edge of top course

$$M = (88{,}000)(15)\ 1{,}320{,}000 \text{ in.-lb.}$$

$$A_s = \frac{1{,}320{,}000}{(16{,}000,)\ (0.875)(28)} = 337 \text{ sq. in.}$$

Use seventeen ½-in. round bars = 3.52 sq. in. each way.

$$\text{Bond} = \frac{88{,}000}{(18)(1.57)(0.875)(28)} = 126 \text{ lb. per sq. in.}$$

This means that additional bars must be used if the bond stress for deformed bars is to be met.

$$\text{The number required} = (18)\left(\frac{126}{100}\right) = 22.7$$

Use twenty-three ½-in. round deformed bars each way to meet the bond requirements.

GRILLAGE FOOTINGS

13. Steel Grillage Footings.—In the early days of the skyscraper and until replaced in recent years by reinforced concrete, steel grillage footings were used very extensively in large buildings to distribute the column loads over relatively large areas with a minimum required depth of footing.

The beams forming the grillage are encased in concrete. This, however, is not assumed as adding to the strength of the footing but rather serving as a protection for the steel beams against rust. The steel beams in the grillage must be designed to resist the maximum bending moment, the shearing stresses, and with a spacing of beams that will readily admit the placing of concrete between them and at the same time allow the concrete filling to distribute the load. Some designers do not assume that the concrete between beams tends to resist any tendency of the webs to buckle under load. It would seem, however, that where properly encased in concrete and with separators placed between beams at frequent intervals, buckling would be impossible. On this basis, the webs must be figured for bearing steel on steel.

The bottom beams of a grillage footing should rest on a bed of concrete not less than 4 in. thick and preferably 6 to 9 in., and be completely surrounded at ends by at least 6 to 9 in. of concrete, while the space between beam flanges should never exceed three times the flange width or be less than 2½ in. to allow proper

tamping of the concrete. The assembled grillage is usually blocked up in position, leveled and the concrete poured around it so as to give bearing on all flanges.

The beams in a grillage should not be painted. To prevent the beams from spreading and to make them act as a unit, gas-pipe separators (not cast iron since they break the continuity of the concrete) should be placed at the ends and under all points where concentrated loads occur. For beams over 8 in. deep, two lines of separators should be used.

The bearing area of a grillage is generally assumed as equal to the length of beams times the width out to out of flange edges, while some codes allow an additional width equal to the width of the upper outer flanges on both sides. This latter additional area being allowed on the basis that the concrete tamped between the flanges will distribute the bearing to the concrete adjacent to the lower outer flanges.

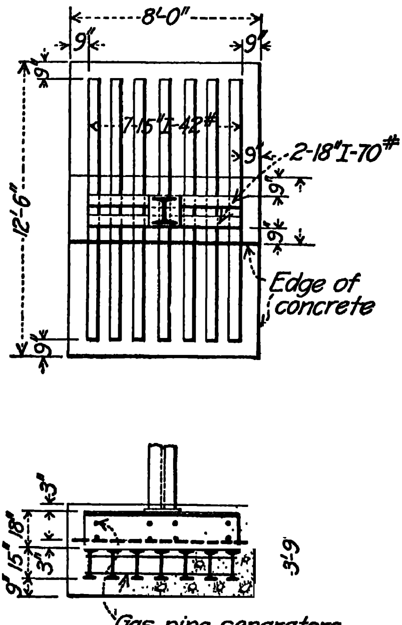


Fig. 20.

Illustrative Problem.—Design a steel grillage footing for a 14-in. Bethlehem H column carrying a total load of 395,000 lb., the bearing value of soil being 4000 lb. per sq. ft.

Area of footing required = 100 sq. ft. For this type of footing the rectangular form will generally be the most economical so a footing 8 × 12 ft. 6 in. will be used (Fig. 20).

A base plate, 16 × 16 × 1 in. will be used to distribute the column load to the first layer of the grillage. Using a bearing value for steel on steel of 20,000 lb. per sq. in., the direct bearing area required

$$\frac{395,000}{20,000} = 20 \text{ sq. in. (approx.)}$$

The length of base plate being 6 in., the total web thickness required for the top grillage is 1.25 in.

Assuming the 9-in. protection of concrete at the ends of the beams effective, the lower layer will be 11 ft. long and 6 ft. 6 in. in width and the top course 6 ft. 6 in. long and 16 in. wide.

The moment on the lower tier of beams

$$M = (4{,}000)\left(\frac{(6.25-0.67)^2}{2}\right)(8)(12) = 5{,}971{,}200 \text{ in.-lb.}$$

The section modulus

$$S = \frac{5{,}971{,}200}{16{,}000} = 374$$

Use seven 15-in. 42-lb. I-beams with $S = 412$.

The shear in each beam

$$V = \frac{395{,}000}{(7)(132)}\left(\frac{132-16}{2}\right) = 25{,}900 \text{ lb.}$$

Each beam will develop (15)(0.41)(10,000) = 61,500 lb. in shear, which is ample.

The moment in the upper tier of beams

$$M = (4{,}000)\left(\frac{(4.0-0.67)^2}{2}\right)(12.5)(12) = 3{,}328{,}000 \text{ in.-lb.}$$

$$S = 208$$

Use two 18-in. 70-lb. I-beams with $S = 205$.

The shear in each beam in the upper tier

$$V = \frac{395{,}000}{(2)(78)}\left(\frac{78-16}{2}\right) = 78{,}430 \text{ lb.}$$

Each beam will develop (18)(0.72)(10,000) = 129,600 lb., which is ample.

The bearing required to take the load from upper layer

$$\frac{395{,}000}{20{,}000} = 20 \text{ sq. in.}$$

The web of each beam in the lower layer has (0.41)(6.26) = 2.56 sq. in. at each intersection. Total bearing = (2.56)(14) = 35.8 sq. in.

The web of each beam in the upper layer has (0.719)(5.5) = 3.95 sq. in. at each intersection, which is sufficient.

14. Timber Grillage Footings.—For temporary construction in buildings on poor soil or for permanent construction where the footings are always under water, timber grillage footings can be used. Where supporting bearing walls, these footings are usually made up of three courses of timber under the wall, the top and bottom courses being 2- or 3-in. planking laid longitudinally, with heavy cross timbers at frequent spacing between them to take care of the cantilever moment developed due to the projection beyond the face of the wall supported (see Fig. 21). The planking distributes the load over the transverse beams and these latter must be designed so that the stresses developed do not exceed the allowable. For temporary work, the extreme fiber stress in bending may be taken at 1,600 lb. per sq. in., and bearing across the grain at 500 lb. per sq. in. The spacing of the transverse beams will be determined by the strength of the planking, and also by the projection of the beams.

For column footings of ordinary size, the load from the column is first transmitted to a sill made up of one or more timbers which in turn transfer the load to timbers laid transverse to the first and resting on planking to guard against unequal settlement of any of the timbers due to inequalities in the soil. Such a footing is shown in Fig. 22. As footings increase in size, additional layers

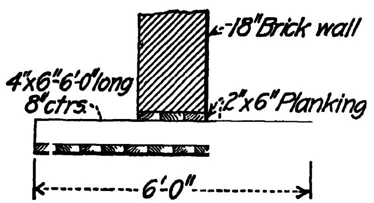


FIG. 21.

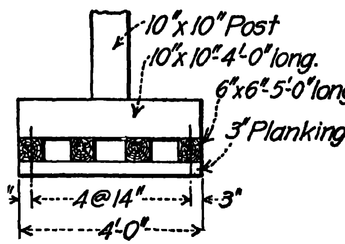


FIG. 22.

of timbers laid at right angles to each other may be necessary to properly distribute the load.

Illustrative Problem.—Design a continuous timber grillage to support a brick wall with a total load of 12,000 lb. per lin. ft., the allowable bearing on the soil being 2,000 lb. per sq. ft.

$$\text{The width of footing required} = \frac{12{,}000}{2{,}000} = 6 \text{ ft.}$$

The wall being 18 in. wide at the bottom, the timber footing will project 2 ft. 3 in. beyond it on each side. The moment developed by this cantilever per lin. ft.

$$M = (2{,}000)(2.25)(13.5) = 60{,}750 \text{ in.-lb.}$$

The section modulus of timber required for this moment

$$S = \frac{60{,}750}{1{,}600} = 37.9$$

Using nominal sizes of timbers, since material need not be dressed, the section modulus $\left(\frac{bd^2}{6}\right)$ for a 4×6-in. timber is found to be 24. For the section modulus of 37.9 per ft. these 4×6-in. timbers will have to be placed 8 in. on centers. Planking 2 or 3 in. thick will be sufficient to distribute the load to beams at this spacing (see Fig. 21).

Illustrative Problem.—Design a timber grillage footing for a 10×10-in. post carrying a load of 40,000 lb. Stresses to be the same as given in the previous example.

Since the allowable bearing across the grain is 500 lb. per sq. in., no bearing plate will be required under the post to distribute the load.

$$\text{The size of footing required} = \frac{40{,}000}{2{,}000} = 20 \text{ sq. ft.}$$

Assume a footing 5×4 ft., the sill under the column to be placed in the 4-ft. span.

Assume the cantilever of the sill to begin at the center of column, then

$$M = (2.0)(5)(2{,}000)(1.0)(12) = 240{,}000 \text{ in.-lb.}$$

Section modulus required

$$S = \frac{240{,}000}{1{,}600} = 150$$

Section modulus of a 10 × 10-in. timber $= \frac{bd^2}{6} = 166.6.$

Use this size of timber for the distributing beam or sill. The cantilever beams below the sill will have a projection of 25 in. and the moment developed for one side of footing will be

$$M = (2.07)(4)(2{,}000)(12.5) = 207{,}000 \text{ in.-lb.}$$

$$S = \frac{207{,}000}{1{,}600} = 130$$

The section modulus of a 6 × 6-in. timber = 36, and four 6 × 6-in. timbers would give a total of 144. Use these spaced as shown in Fig. 22 at 14-in. centers with 3-in. planking underneath laid tight over the entire area.

In timber grillages all timbers should be secured to each other with wooden dowels, or iron drift bolts.

TRANSFERENCE OF COLUMN LOADS TO FOOTINGS

15. Methods.—The load carried by a column may be transferred from the column to the footing by (1) direct bearing or (2) by a combination of direct bearing and bond stress developed on bars embedded in the column and the footing.

16. Cast Iron and Wood Columns.—The loads on steel, cast iron, or wood columns can only be transmitted to a column footing by direct bearing, the requirement being that the column must have a base large enough to keep the bearing on the footing material (usually concrete) within the allowable. For wood columns the base plate may consist of a steel plate of sufficient thickness to resist the bending due to the projection beyond the face of column, a cast iron base, or a fabricated steel base. For steel columns carrying relatively light loads, the column base is usually made an integral part of the column by riveting angles and plates to the bottom of the column. When very heavy loads are carried, steel billet slabs, steel or cast-iron castings, or grillages of steel beams, should be used. These should be designed so as to allow the blocking and leveling up of the base and anchoring these to the footing. The space between the bottom of the base and the concrete footing is then filled with cement grout. This cannot be done with steel slabs and the grout must therefore be spread over the footing top, leveled off, and the steel slab set and leveled thereon.

$$\text{The area of base required for a column} = \frac{\text{Total column load}}{\text{Allowable bearing}}.$$

The bearing stress on the concrete is therefore the determining factor. Good practice as exemplified by the Joint Committee Report allows a stress in bearing of 25 per cent of the ultimate, or 500 lb. per sq. in. for 1:2:4 concrete and 600 lb. per sq. in. for 1:1½:3 concrete, except that when the area of the top footing is at least twice that of the column base the allowable stress in bearing is 35 per cent of the ultimate, which amounts to 700 lb. per sq. in. for 1 : 2 : 4 concrete and 840 lb. per sq. in. for 1 : 1½ : 3 concrete. This provision usually makes it more economical to use a larger top course on the footing and design it and the column base for the greater unit pressure.

17. Masonry or Concrete Piers.—The loads from brick, stone, or plain concrete piers can be transmitted to the footings only by direct bearing, the allowable bearing value of the pier materials being the limiting factors.

18. Vertically Reinforced Columns.—For vertically reinforced columns of concrete the bearing stress, considering the entire area of column effective, will usually be found to be less than the allowable bearing stress on the footing concrete and the load in the column is, therefore, considered as transferred to footing by direct bearing with a few stub bars placed in the footing and lapping with the column bars to anchor the column to the footing.

19. Spirally Reinforced Concrete Columns.—The allowable unit compressive stresses in spirally and vertically reinforced concrete columns being relatively high, the critical section for bearing will be the top course of the footing. In some cases the average bearing over the gross area of the column base will be within the allowable for the top of the footing and the design can be made for direct bearing only.

For heavily reinforced columns of rich concrete (1 : 1½ : 3 or 1: 1: 2 mixtures) the unit stresses on the column core will be found so high that the average bearing over the gross area of the column will exceed the allowable for 1: 2: 4 concrete and therefore the load must be transferred partly in bearing and partly by bond on the stub bars placed in the bearing and partly by bond on the stub bars placed in the footing. In some cases the load may be transferred by direct bearing by putting in a top course of a richer mixture of concrete. Some designers make use of a spirally reinforced cap to transfer the load to the footing, designed

in the same manner as the column but of larger diameter. This method, however, is rather uncommon in practice and not as economical as employing a richer mixture for a larger block of plain concrete with stub bars to transfer some of the load by bond.

In arriving at the number of stub bars necessary for transference of load partly by bond and partly by direct bearing, the amount of load transferred by bearing should first be determined. This load (P_1) equals the area of column multiplied by the allowable unit bearing. Then the load transferred by bond

$$P_2 = \text{column load} - P_1$$

The number of bars required for bond

$m = \dfrac{P_2}{uo33d}$ for 1:2:4 concrete and $m = \dfrac{P_2}{uo26d}$ for 1:1 ½:3 concrete

where d = the diameter of the bars used.

If this computation shows that more stubs bars are required than the number of vertical bars in the column, then a richer mixture should be used in the cap, thus increasing the bearing value and cutting down on the number of stubs required.

SECTION 5

UNDERPINNING

By Edmund Astley Prentis, Jr.

Underpinning is the art of installing new foundations, either in lieu of old ones, or under them, and is distinctly an art, rather than a science. Therefore, the methods about to be described must be interpreted with practical experience and applied to each underpinning problem after its careful study.

The underpinning of a structure may be divided briefly into two operations—namely, the support of the structure during the installation of the new supports and the installation of the new supports themselves, or the underpinning proper.

1. Support of Existing Structures.—This portion of the work especially requires good judgment and experience in order that no damage may result and that the expense of the operation may be as small as possible. From the necessities of the case only very general information can be given.

Buildings are much stronger than are commonly supposed. They are usually designed with a large factor of safety, and correspondingly can stand more abuse than offhand one would imagine.

It is only necessary to recall pictures of bombarded cities on the continent to understand the tremendous amount of punishment a building can stand without collapsing. It was no uncommon thing to see a building stand apparently against the laws of nature (see Fig. 1).

Of course, buildings cannot stand in defiance of these laws and any thoughtful engineer can understand the arching action which comes into play automatically when necessary. For instance, consider an ordinary case—a six story building, 50 ft. wide and 100 ft. long. Assume it is necessary to underpin the long side of such a structure resting on a mixture of sand and clay. As built, it is resting on a spread foundation uniformly loaded. The moment a small excavation is made under any part of this wall, the load formerly carried by that part of the soil is carried

to the ground on either side of the excavation, as shown in Fig. 2, by the arching action of the wall.

Of course, this is almost the simplest example that can well be magined but the principle is the same in more complex cases.

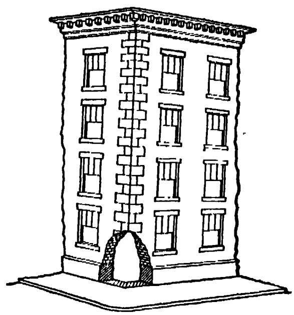


Fig. 1.—Building with portion of wall removed showing arching action developed.

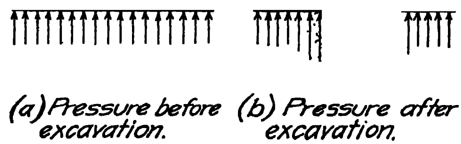


Fig. 2.—Length of arrows indicate relative intensities of soil loads.

In the example cited, assuming a slight settlement of the structure permissible, no additional support would be required during the underpinning operations.

1*a*. Shoring.—When additional supports are required in underpinning, the easiest and most commonly used are called

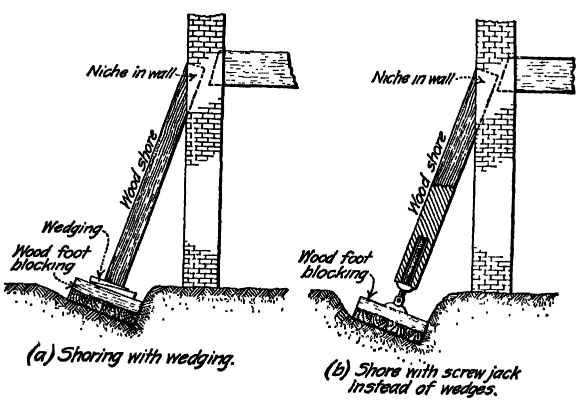


Fig. 3.—Typical shoring.

"shores." These, as shown in Fig. 3, are usually long wooden posts placed in an inclined position against the wall of a building in suitable niches in the masonry and adequately supported at the bottom, usually by a wooden crib. It is good practice to

keep shores as nearly vertical as is expedient in order to minimize the side thrust against the wall. It is also advisable to place the head of the shore opposite a floor line in order to minimize the danger of pushing the wall in.

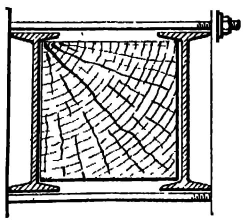


FIG. 3A.—Method of reinforcing wood shore for heavy loads.

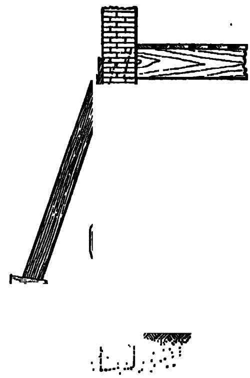


FIG. 4.—Shoring with support for wall below shore.

For heavy loads, care should be taken to have the head of the shore bear uniformly in the masonry work. Sometimes grouting with cement is the easiest thing to do and the wedges should always be well and truly driven until the shore is carrying the load desired. It is sometimes well to use iron wedges inserted between plates at the foot of the shore, instead of wooden ones, and the shore itself may be reinforced by means of steel channels or I-beams bolted to it, as shown in Fig. 3*A*.

Attention should be called to the fact that the old proverb—"A chain is as strong as its weakest link"—applies to a shore, and usually its weakest link is either its footing or the niche in which it rests so that care should be taken to have them properly proportioned.

When shores are used, the foundation is relieved, of course, of part of its load, but the only load carried by the shore comes from the part of the structure above its head—except so far as the tensile strength of the structure comes into play, which, in the case of brickwork, is a very small amount. Shores may be combined with needles (described in Art. **1b**), the needles carrying the part of the wall from head of the shore to the ground. However, for this work the combination shown in Fig. 4 has been used to advantage.

1b. Needling.—Another method of temporarily carrying structures is by "needling," which merely means that temporary beams called "needles" are installed to carry the

structure until the underpinning is completed. I-beams are generally used, the sizes can be easily picked out by computing the loads, span, etc., with the aid of any of the standard steel handbooks. Their use is shown diagrammatically in Fig. 5.

There are several points to be noted in needling. In the first place when carrying brick walls or piers, it is wise to put in wood fillers above the beams, as these crush while the wedges are

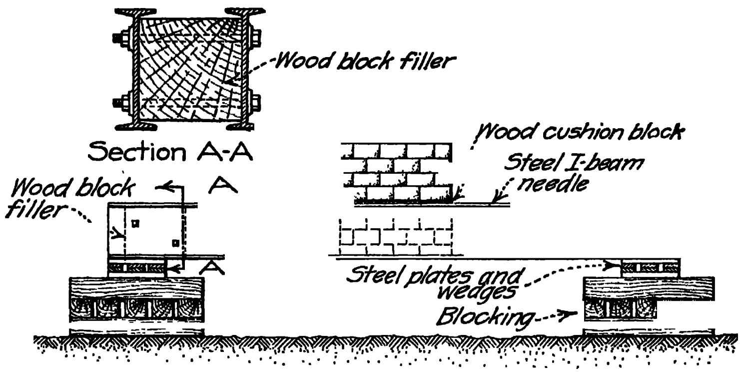


FIG. 5.—Simple needling operation.

being driven in, thus putting a uniform bearing on the masonry and helping to prevent it from acquiring a crushing stress at the edges.

Then, as with shores, particular attention should be given to the footings carrying the needles. These should be adequate, otherwise, when the needles are wedged up, the footings will settle as the needles acquire the load and necessitate rewedging.

A third thing to guard against is the possibility of the needles flopping over on their sides while in use. This particularly applies to I-beams and can be prevented by using beams in pairs with wood fillers and lashings, as shown in Fig. 5.

The various combinations that can be resorted to are limitless —wooden towers instead of crib-work for the supports of the ends of the needles can be resorted to or even temporary concrete foundations in important cases. Two beams might be concreted together for a short distance at each end in order to prevent their wobbling.

Fig. 6 shows the needling used to carry 10 × 10-in. cast iron columns of a ten story building with column loads of 250 tons each. The needles were carried by wooden towers resting on a continuous row of 6 × 12-in. timbers, 6 ft. long. In this particu-

lar instance there was a serious problem in devising a suitable grip in the cast iron column and the one devised is shown in Fig. 6.

2. Strengthening and Supplementing Existing Foundations.—In many cases, shores, needles, and their combinations are not

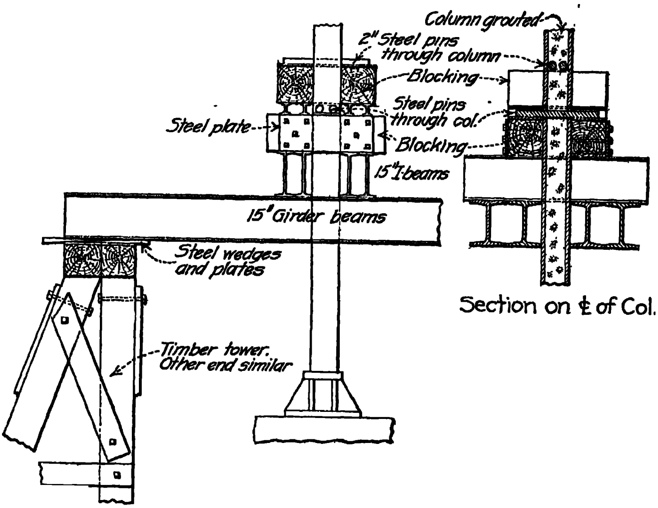


Fig. 6.—Needling for 12-in. cast-iron column. Load, 250 tons per column.

needed, though often it may be necessary to strengthen or supplement the existing footings before excavating beneath them.

In the old days buildings were often erected on dry rubble walls. This was done even on structures of considerable magnitude—for instance, on the City Hall of Philadelphia. Many contractors when handling such a condition point up the opening with the vain idea that the footings are thus strengthened. The very defects of such a construction are a help. All that is necessary is to scrape the dirt out of the joints, put up a tight form and pour in very liquid grout, which will, in most cases, effectually bond all the stones together and make good masonry out of bad.

Sometimes in the case of a footing wall of poor rubble masonry, reinforcing rods can be used to advantage.

Instead of rods, I-beams can often be substituted to advantage, by cutting holes 2 or 3 in. in diameter in the webs in order to prevent voids in the concrete or mortar. Typical uses of these methods are shown in Fig. 7.

Very often when underpinning columns of buildings it is economical to join them together with continuous foundation slabs

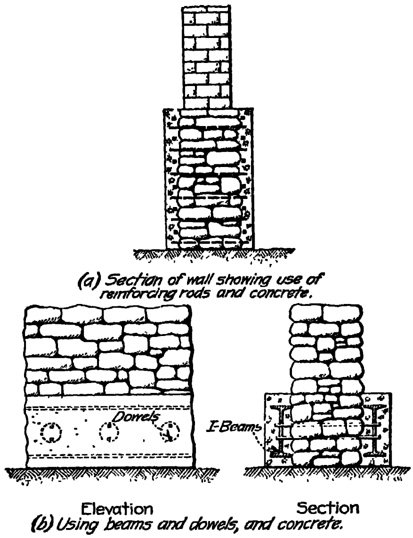


FIG. 7.—Strengthening old walls.

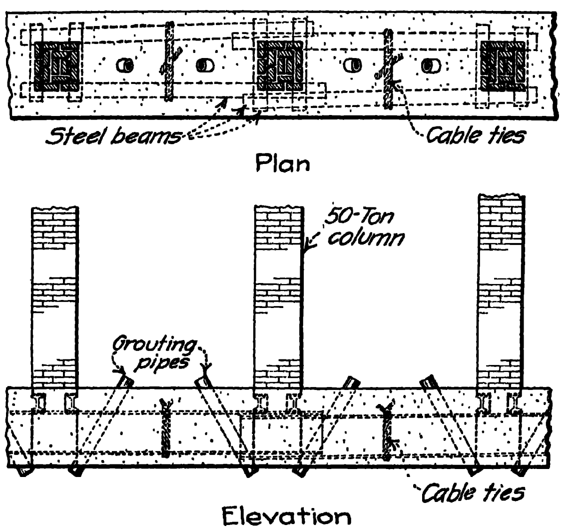


FIG. 8.—Foundation reinforced with grillage for underpinning.

called "grillages." This not only ties the footings together, but also gives a large spread foundation which carries columns while the excavation is going on underneath them.

Many ingenious combinations can be made generally using steel and concrete, the steel being in the shape of I-beams as well

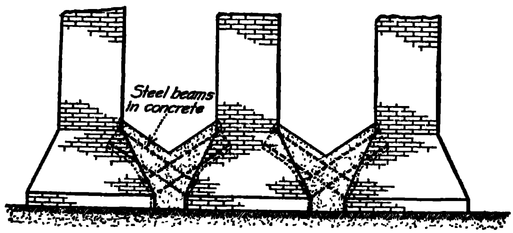


Fig. 9.—Foundation reinforced for underpinning without grillage.

as reinforcing rods. Second-hand steel is generally used and generally is of such size as to fit the local conditions in the field rather than computed sizes.

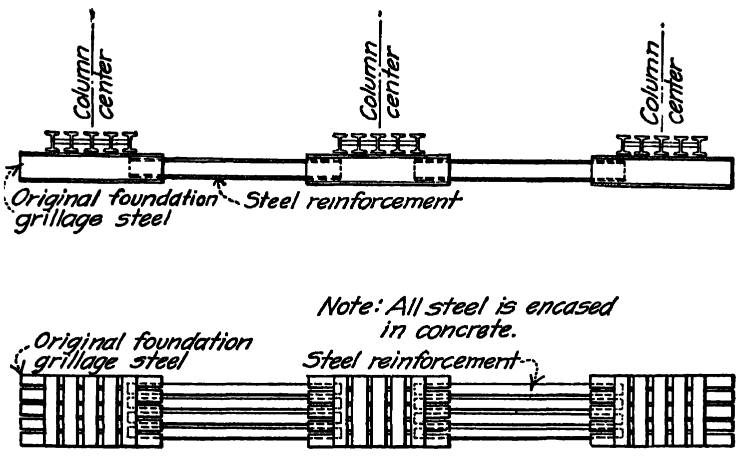


Fig. 10.—Reinforcing steel foundation of modern building.

Fig. 8 shows such a grillage, including ties (usually a tourniquet of steel wire cable), and grout pipes for pouring in grout to make a snug joint between grillage and new underpinning when installed.

Another application of steel and concrete for reinforcing foundations of an extremely heavy old-fashioned building is shown in Fig. 9.

For more modern buildings, grillages may be reinforced, as shown in Fig. 10.

The above types of reinforcement are merely to give some idea of how some previous problems have been solved. The number

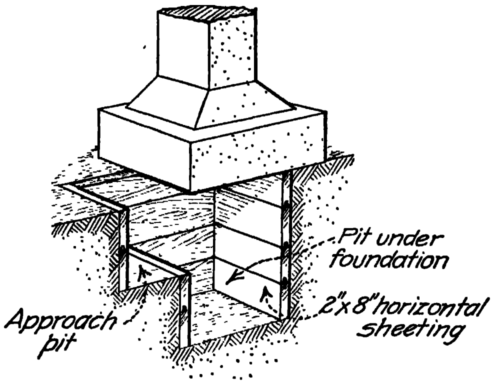


FIG. 11.—Foundation with underpinning pit.

of combinations to be used is limitless and much depends on the ingenuity of the constructor.

3. Underpinning Proper.—Assuming now from the foregoing that the building is in proper condition to have the underpinning installed, it is well to direct the attention to two important points.

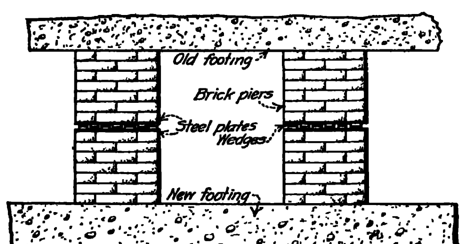


FIG. 12.—Wedging in brick underpinning piers.

The first is that every effort should be made not to lose or loosen the earth or ground beneath the footings. The second is that it is of primary importance to wedge up properly between the underpinning and the structure—in other words, to actually put the underpinning to work by bringing the load to it. If special care is not taken in this regard, the load will be carried to the underpinning subsequently by settlement of the structure itself.

When moderate loads only are being handled in connection with needles and shores, the wedges may be driven until they are relieved of all or part of their load. With heavier loads this cannot be done and the work should be so installed as to regulate the ensuing settlement so as to have it uniform. When the work is properly handled, even in the most difficult cases, settlements should usually not exceed ¼ to ½ in., and settlements of over 1 in. are likely to prove troublesome.

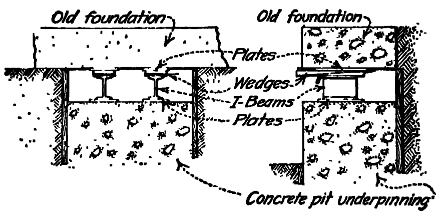


Fig. 13.—Wedging of a typical underpinning pit.

Underpinning usually consists of pits filled with masonry or piles, or combinations of both. Pits are generally used where possible, the limiting factor usually being water level. They may be dug to the required depth with or without sheeting, depending on the quality of the soil. Where sheeting is required, the usual vertical tongue-and-groove sheeting may be used if enough headroom is available. Usually, however, it is more economical to use horizontal sheeting, say 2 × 8-in., as shown in Fig. 11.

When pits are completed, they are filled with brick or concrete and, when set, are ready for wedging.

In case a light wall is being underpinned, perhaps with a brick or concrete wall, wedging stones were formerly used. Now steel plates and wedges are more commonly used, as shown in Fig. 12.

A very convenient method of wedging is by the use of short pieces of I-beams either resting on end or on their flanges and wedged with steel wedges, as shown on Fig. 13.

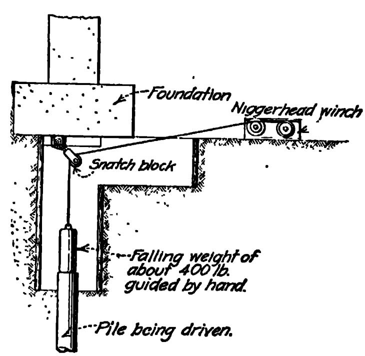


Fig. 14.—Driving pile with falling weight.

4. Piles Used in Underpinning.—Piles used in underpinning are generally steel shells, driven in the ground, usually in short sections, excavated, and concreted. They are used generally when it is necessary to go below ground water level,

to avoid the possible loss of ground, or the use of pneumatic methods. The diameters and thickness of shells vary widely. Sometimes piles are driven 3 ft. or more in diameter using compressed air as in caissons, having a man in the working chamber. Generally, however, the sizes are smaller, varying from 10 to 16 in. in diameter and in thickness of shell from $\frac{7}{64}$ to $\frac{3}{8}$ in. using sleeve connections.

In case headroom is available they may be driven down by means of a winch and a falling weight, as shown in Fig. 14, or by means of one of the many pneumatic hammers, such as the McKiernan-Terry.

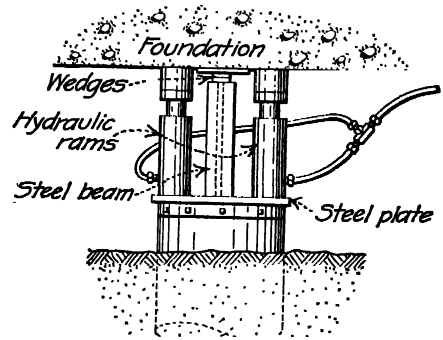


FIG. 15.—Method of driving pile by hydraulic rams. This illustration also shows method of wedging. "Pretest Method" (patented).

Generally, however, the headroom is limited from 3 to 7 ft., in which case 1 or 2 ft. sections of pipe are driven by means of hydraulic rams, as shown in Fig. 15.

These hydraulic jacking outfits are very efficient though requiring careful mechanical attention and a good bit of discretion in their use because of the large upward thrust they give when the pile offers sufficient resistance. The working pressures run up to 5000 lb. per sq. in. It is rather unwise to use higher pressures because the leather gaskets become so compressed that they then have to be frequently renewed, which is an inconvenience. The most common diameter of the rams is $4\frac{1}{2}$-in. and, with this size, each 1000 lb. on the gage indicates a thrust of about 8 tons, or, with 5000 lb. per sq. in., a 40-ton reaction. For thrusts greater than this, two rams are connected up in parallel and with 5000 lb. pressure each would, of course, give 80 tons thrust. With these great loads easily obtained, it is obvious care must be used not to damage the structure which is used as a reaction.

As the pile is jacked into the ground, resistance to its progress increases rapidly. Every few feet the earth is excavated from within the cylinder. Earth augers and miniature orange-peel buckets attached to suitably jointed rods are used, also jetting,

and with the pile excavated, the resistance to driving generally disappears. A convenient rig sometimes employed consists of putting the suction of a diaphragm pump down the pile, keeping the pile full of water and jacking at the same time. In this method progress is more or less uninterrupted by the excavating process but care must be taken not to let the suction of the pump get too near the bottom of the pile as this would cause a loss of ground.

The skin friction of these cylindrical piles is always very small, around 50 lb. per sq. ft. of surface—in other words, a 25-ft

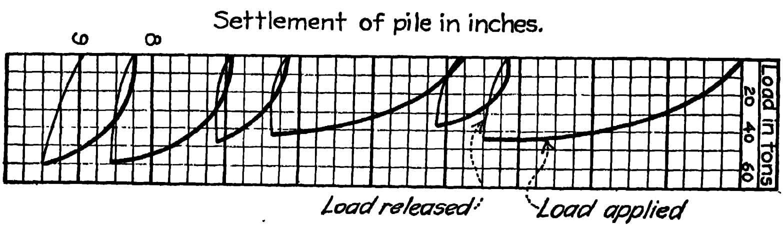


FIG 16.—Loading test on jacked pile.

pile, 14 in. in diameter, requires only 4 or 5 tons to thrust it into the ground, provided, of course, that the pile is excavated to within a few inches of its bottom.

When driven the required depth, and excavated, the pile is then concreted, using bottom dumping buckets, if necessary. The hydraulic rams are again placed on the cylinders and the piles tested by reapplication of the load, well in excess of the load the pile is designed to carry. In coarse sand or gravel such a pile will give a reaction of 80 tons with a settlement of a foot or two, or with greater settlements in finer grained soils. The reason for this is that the soil at the base of the pile is compressed, forming a bulb. While the pressure is maintained, this bulb will generally remain intact but is partially or entirely destroyed if the load is lost, so that further settlement would be required for the pile to sustain its load. A settlement curve of such a cylinder is shown in Fig. 16.

To prevent this resettlement a patented method called the "Pretest Method" is used. By this method the pile is wedged up by means of I-beams and wedges while the load is still on the pile, and when the jacks are removed an enormous load is immediately taken up by the beams because of the attempt of the

pile to rebound when the jacks are removed. This is shown in Fig. 15.

If this method is not used, after testing the jacks are removed and the piles wedged up by means of beams, plates, wedges, etc.

5. Design of Underpinning.—The design of underpinning is based on the same general principles that apply in foundation work.

In general, soil loadings of from 1 to 4 tons per sq. ft. apply on floating or spread underpinning, and piles are assigned loads of from 25 to 40 tons each, depending on soil conditions—though higher values may be used if rock can be reached.

SECTION 6

FOUNDATIONS REQUIRING SPECIAL CONSIDERATION

DEEP BASEMENTS AND MACHINERY PITS

BY H. S. BAKER

The construction of deep basements and pits involves on a small scale all the methods of foundation construction from open unsheeted excavation to the use of cofferdams and caissons. This is especially true in wet ground where such construction is almost always troublesome and expensive. This kind of work is slow at the best and many progress schedules which seemed liberal to the designing engineer have been entirely disarranged by water under the ground.

Ground water is not detected by the common method of wash borings, test pits being much more reliable in this respect. Wherever deep pits are contemplated, especial attention should be paid to this point in the preliminary explorations and, if water is encountered, it is always advisable to give serious consideration to the possibility of omitting the pits if the same purpose can be served by a modification of the design. This may mean the raising of the basement floor, or perhaps all the floors of a building, or the adoption of a different type of conveying machinery.

After it is determined that the pits are a necessary part of the building, their design should be worked out carefully with regard to possible construction methods and the necessity to make them tight and waterproof. They must be borne in mind also in preparing cost estimates and progress schedules.

1. Design.—During all stages of the design, underground conditions and construction methods should be considered. If the ground is dry, concrete walls (plain or reinforced) to suit the depth can be supported on ordinary footings; floors can be laid after the walls are finished.

The careful designer can often make considerable savings by observing closely the behaviour of test pits to determine the proper angle of repose for the earth which he has to support.

Particularly in the more shallow pits, engineers are often extravagant with reinforcing steel in walls and floors. Architects and builders who work by the accumulated experience of the building craft rather than by mathematical analysis seem to understand the strength of plain masonry walls better than many engineers and their confidence is justified by thousands of successful examples.

In wet ground, walls and floor are preferably monolithic and usually thicker than required to withstand earth pressure and the loads to be carried. If a special system of waterproofing is to be used, the method should be determined early and shown on the plans.

2. Construction.—Basements and pits in dry ground are no different from any other form of foundation construction. The excavation, bracing, and concreting are done by the ordinary methods and, with correct design and careful construction, good results are certain. Pits in wet ground are an entirely different matter. The ordinary foundation troubles are magnified by the fact that instead of digging a hole to be filled up with good ordinary concrete and earth, the builder is making a box with comparatively thin walls which is to remain permanently open and is to house expensive machinery and goods which must be protected from water at all times. Sometimes the most careful builders do not secure watertight pits and large sums have to be expended for waterproofing the finished work with more or less satisfactory results. This is usually due to failure to appreciate the difficulties during the period of design and to starting the work without adequate preparation and equipment.

Pits of any considerable size should be sheeted solid from near the top unless the ground is unusually firm and remains so upon exposure to air and water. This sheeting should be thoroughly braced. A careful and experienced contractor usually knows how to do this without any instruction, using good judgment based on experience, but it is well for the young engineer to check up the sheeting and bracing by calculations of earth pressure and to make a careful daily inspection of the sides of the hole. Danger can usually be detected by deflection of the timbers and movement of the walls. In this way the engineer will also add to his theory practical knowledge of the lateral pressure exerted by various kinds of earth. The most experienced are apt to be the most careful in this respect as there is nothing much worse on a

job than a big hole filled with caved-in earth, bracing, and sheeting with possibly some men buried under it.

Perfectly dense concrete is watertight even when exposed to a head of more than 20 ft. of water. Dense concrete can be made from well graded aggregate and good cement well mixed with the right proportion of water and placed in tight forms which are clean and free from running water. The above conditions can all be assured by ordinary careful work except the last one, which is sometimes very difficult. If a pit can be kept dry while the concrete is being placed, and until it has thoroughly set, a watertight job can be practically guaranteed. If water is allowed to run or seep through the green concrete, leaks are almost certain to show. The problem is then: how to keep the pit dry until the concrete has set.

When conditions are not too bad, this can often be done by ordinary pumping from a sump. The sump should be located where it can remain in service until the concrete is set. Sometimes the sump has to be dug outside the line of finished work to a depth below the subgrade. If water flows across the bottom of the hole, it is sometimes possible to dry the subgrade for the concrete by digging it 6 in. or a foot lower and backfilling to the proper elevation with crushed stone or coarse gravel through which the water can flow to the sump without disturbing the concrete above.

Tile drains are sometimes laid on the subgrade to lead the water away and they can usually be plugged at their connection with the sump after the floor has set hard. There are a number of basements in Chicago where such sub-drains were used and the floors were not tight enough to withstand the water when the pressure increased after these pipes were plugged. In these cases the drains were left permanently connected to the sumps. This method is not recommended but the basements are kept perfectly dry at the cost of continual pumping.

At the Samson Tractor works in Janesville, Wisconsin, in 1919, an ash pit and conveyor pit had to be sunk 10 ft. below the basement floor and nearly the same distance below ground water level. The soil was coarse sand and gravel. Several large pumps were installed and a tight cofferdam constructed and driven down as the excavation progressed. When the bottom was reached, the flow of water amounted to several thousands of gallons per minute. The plans showed a 12-in. floor but the

excavation was carried 2 ft. deeper and 2 ft. of concrete was poured all over the bottom except for a channel around the outside left to conduct the water to the sump. When this floor had hardened, little springs of water came up through it in a number of places and it was apparent that the top floor would be no better unless it could be protected from the water while soft. A large piece of heavy canvas was cut to fit the floor and to turn up 18 in. all around in the walls. This canvas was soaked in paraffin oil and was spread on top of the sub-floor. Floor reinforcing rods were placed. Wall forms and reinforcing

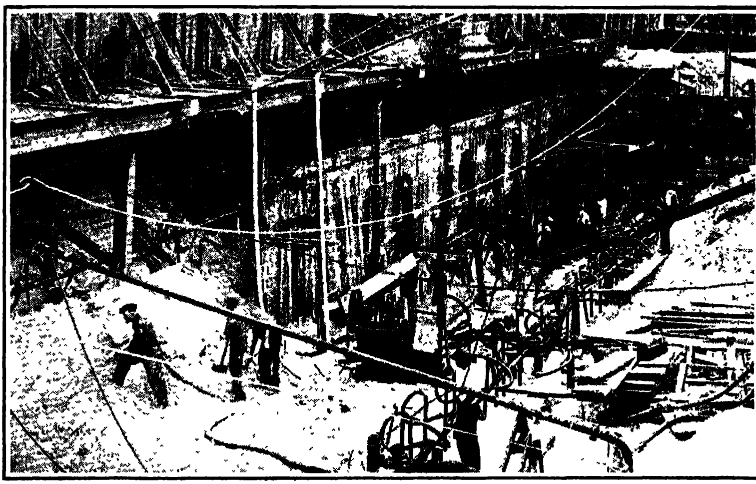


Fig. 1.—Lowering ground water by the use of well points.

bars were set up and the waterproofing sheet was carefully turned up in the center of each wall. The concrete of the top floor was poured on top of the canvas and the walls were poured monolithic with the floor. This pit was practically dry when completed and was better than any of the other pits at this location which were constructed by different methods.

The neatest method of drying a pit of this kind is the method of lowering ground water by the use of well points. By this method well points are driven down around the area to be dried and are connected by pipes to a pump which is kept in continuous operation. The first application of this method of which the writer knows is recorded in the report of the Metropolitan Sewerage Commission for 1901. It was used on Section 69 by Beckwith and Quackenbush of Mohawk, N. Y. as follows:

"For a length of 300 ft. near Mount Hope Station, the fine sand in which the lower part of the trench was excavated moved under the head of ground water. Five tubular wells were driven through the stratum of fine sand into the underlying gravel 19 ft. below. Pumping was continued for about a month when the soil was dried enough for the masonry to be placed."

This method was used on a large scale in all kinds of underground work during the construction of the city of Gary, Indiana.

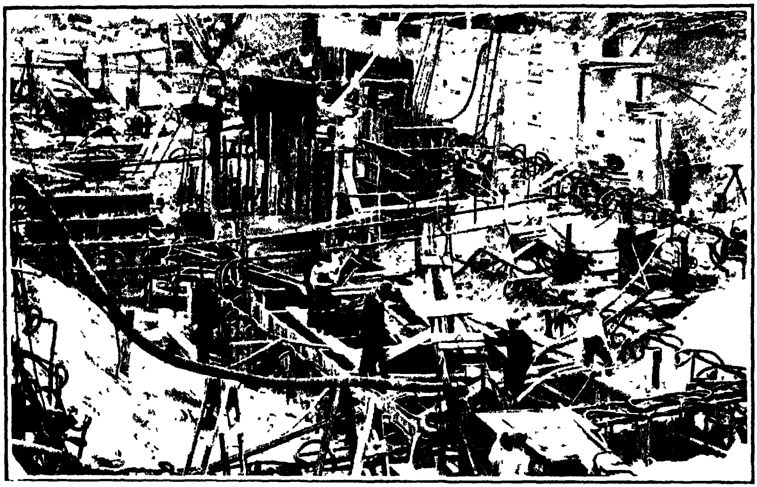


Fig. 2.—Lowering ground water by the use of well points.

It has been extensively used on sewer work about Chicago with uniform success. During 1921 it was used for the basement of the Cornelia Garage on Broadway in Chicago and was successful as far as it was possible to use it. The south basement wall was adjacent to the next building and it was necessary to remove all the points on one side before the floor could be placed. The other single row of points was not sufficient to keep down the water and other means had to be adopted to finish the job.

The best known instance of ground water lowering by the use of well points is in excavating for the foundations of the Ambassador Hotel and the Ritz Carlton Hotel at Atlantic City and described as follows in the *Eng. News-Record* of January 13, 1921 (see Figs. 1, 2, and 3):

In both cases the sand was excavated in the open without sheeting to depths exceeding 15 ft. below mean tide in spite of the fact that in one case the work was situated within 100 ft. of the water's edge. This

result was made possible by encircling the work with lines of driven wells and pumping down the water in the sand to a level below the floor of the excavation. The fineness of the sand and its stability when dry made the method particularly effective as the pumping capacity required was moderate, changes in ground water level were comparatively slow, and the excavation stood with vertical banks that showed no tendency to crumble.

In applying well point pumping to the Ambassador a single cofferdam was constructed around the whole site by encircling it with a ring

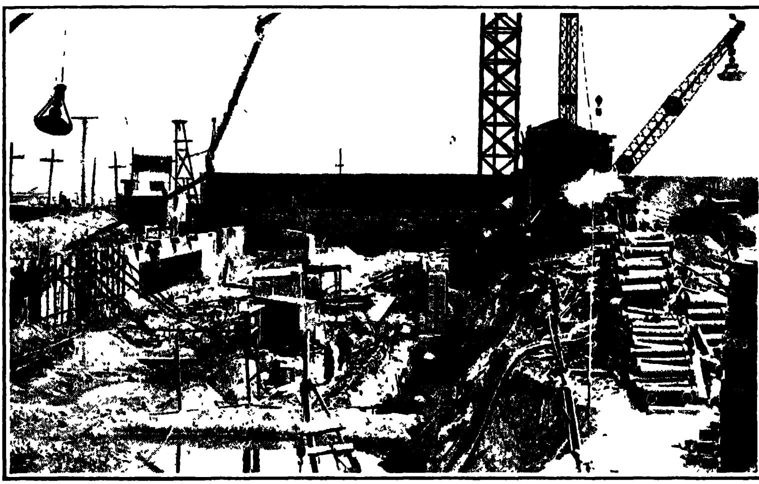


Fig. 3.—Lowering ground water by the use of well points.

of wells pumped as a single unit. The Ritz Carlton work is distinguished by extensive subdivision of pumping and construction work through the use of many local rings of well points forming separate cofferdams. This expedient permitted progressive working from one end of the site towards the other, gave remarkable flexibility and adaptability to changing conditions, and, together with the practice of lowering the ground water in successive stages by using several tiers of wells as excavation proceeded, gave marked economy in both plant cost and pumping cost.

The wells consisted of lengths of 1½-in. pipe fitted with well points 30 to 36 in. long wrapped with 60-mesh screen, which were jetted down into the sand at intervals of from 3 or 4 ft. to as close as 18 in., usually closest in the lowest tier. The top of each well pipe connected through 1½-in. wire covered hose to a 4-in. steel pumping main, to which were connected one, two, three or four electrically driven pump units of capacity from 75 to 200 gal. per minute discharging by pipe line to

the beach. Each of these units was mounted on a platform to form a portable unit that could be shifted around by the derrick.

In jetting down a well point a 1½-in. pipe supplied with water at 40-lb. pressure was pushed down into the ground and moved around or churned as necessary to loosen up an area large enough for the well pipe; the well pipe with point was then dropped into the soft sand loosened by the jet.

Generally the points were put down to depths of 10 to 20 ft. However, the details of the well point work throughout as to depth, spacing, pumping capacity, and the like, were fixed as experience dictated, the system adapting itself flexibly to any required changes. When a particular area of the site was to be excavated to a lower level, for example, a ring of points was sunk around the area, at possibly 4 ft. spacing, to 5 or 6 ft. depth, and pumping and excavation were started simultaneously. If these wells did not draw down the water rapidly enough, or if the ground proved wet in excavating, a set of points was jetted down between those already in place. If the pumps were not able to hold the vacuum, another pump was cut in. Hose and pipe connections were checked for tightness frequently, this being vital to good results. When the excavation was down 4 or 5 ft. a set of points was jetted in around the bottom of the pit to lower the water level farther, excavation then continuing inside this smaller ring.

On the whole, experience at this site was that the ground water level could be lowered 10 ft. in about 4 hr. (with 10 to 12 ft. of lift and about 30 ft. from foot of well to discharge). The return of the water was relatively slow, so that when pumping was stopped it would be half an hour before water came up through the floor of the excavation. The pumpage is fairly represented by the experience with the longest continuous line of wells, whose total length was something over 300 ft. and the pumpage averaged 300 gal. per minute. A vacuum of 12 to 20 in. of mercury was carried at the pump. This was at a distance of less than 200 ft. from the edge of mean tide.

Work was started at the beach end of the site, first on the right hand half of the front wall, the other half being started only after the deep wall footing of the first half had been constructed and the principal hazard of interruption of the pumps was over. Then the side wall and parts of the rear wall were taken in hand. When the next sections of side wall were started, the entire front section was cut off by a line of wells and general excavation of the forward portion continued down to floor level. Underdrains, concrete in base, waterproofing, and final concrete floor were then put in, which took care of further drainage in this section and released the wells and pumps.

It is not always practicable to use this satisfactory method of drying a pit and some other method must be adopted to prevent

water from flowing through the green concrete of walls and floor. The caisson method is sometimes used on a small scale by constructing a tight box of dressed and matched lumber having walls and floor which is set down in the pit and the concrete poured within it using the box as an outside form. Sometimes a box of sheet steel is used for small pits such as those for elevators. This is a reliable method where practicable. The reinforced concrete caisson has been used with marked success for large pits and there seems to be no good reason why it could not be used equally well for small pits by weighting down the caisson sufficiently.

Sometimes the bottom may be sealed by placing a thick mat of dry mixed concrete all over it. The aggregate and cement are mixed without water and the water is drawn up through the mass by capillary attraction. If the flow of water from the bottom is too violent to allow this treatment, the water may be allowed to rise to its normal level and the mat of concrete poured in still water through a tube or be placed by a dumping bucket. The water can be pumped out after the concrete is thoroughly set and the bottom will usually be sealed.

The application of mixtures, membranes or other coatings for waterproofing is discussed in the chapter which follows. The grouting method of waterproofing is practically a part of the concreting and has been used in many tunnels and is applicable to basements. It was used with marked success on the tunnels of the New York Aqueduct. Sheet iron pans were placed against the wall of the excavation to catch the infiltering water and to lead it to a drain pipe extending out through the face of the form. Grout pipes also extended into the space between the pans and the outer wall. The wall was concreted in the ordinary way and the green concrete was not disturbed by water which was intercepted by the pans and carried away through the drain-pipes. After the concrete had thoroughly hardened thin grout was forced in to fill the voids by the pressure of compressed air. By repeated applications of grout, remarkably good results were attained and in most cases the walls were left entirely dry. Grouting was also practiced on the La Salle Street and Washington Street tunnels in Chicago to dry finished walls which leaked. Holes were drilled through the walls and a grouting nozzle inserted with a collar which was jacked tight against a thick gasket which surrounded the nozzle. Grout was then forced in behind the wall and upon hardening it effectually sealed the leaks.

WATERPROOFING OF SUBSTRUCTURES

By Earl G. Swanson

The question of waterproofing, especially of the substructure of a building, is a subject which has been given comparatively very little consideration by the majority of architects, engineers, and contractors. The natural result is that in many cases some system is used that does not fit the conditions.

Previously there has been little need of waterproofing as most of the buildings have been put up almost entirely above ground and that which was below received no particular attention. Today the price of land, particularly in the large cities, is such that basements must be used. Since some of these are below water level, some provision must be made to keep them dry. As most of the foundations of today are being built of concrete, this discussion will be limited to the waterproofing of that material.

It is almost generally conceded that concrete can be made water-tight without the aid of waterproofing agents provided (and there is the whole secret) that 100 per cent workmanship be obtained in all steps. The fact that we often see examples of concrete that are not as good as can be made shows that the personal equation cannot be neglected. It also proves that results cannot be guaranteed without some special agent. Every waterproofing system has its value but some of them are not applicable to certain cases; the choice therefore depends entirely upon the conditions to be met.

3. Methods of Waterproofing.—In general there are three main classes of waterproofing; the integral method, the membrane method, and the surface coating method.

4. Integral Method of Waterproofing.—The integral method consists of incorporating in the concrete some material, either as a paste, powder, or liquid. This material is added during the mixing and the theory is that it will act as a void filler. It is because of the voids in concrete that leaks occur; if these are filled there can be no leakage. If an integral system is used on a job, strict superintendence must see that all precautions are taken to secure an A-1 concrete. It must be remembered that a void filler will not prevent construction joints or porous spots, both of which are the results of poor workmanship. It is possible to secure waterproof concrete without a waterproofing agent if extreme precautions are taken; as this would involve a large expense, the integral filler is used to reduce the cost.

A subdivision can be made in the integral method: inert fillers, water repellants, and chemical combinations.

Among the inert fillers, hydrated lime is the best known. Probably the most valuable advantage of it is that it makes possible the use of a wetter mix of concrete than ordinary, without losing any of the finer particles of the cement or sand. In this way it makes the concrete "fat," making it more workable. It is of special value in chuting concrete, preventing it from sticking to the chutes.

Clay is another inert filler. The difficulty with this material is that it decreases the strength of the concrete.

The water repellents generally have, as a base, lime to which is added a percentage of fatty acids. The theory is that the acids react with the lime to form a lime soap. This is not easily soluble in water and therefore tends to repel it. Heavy oils have also been used with more or less success.

In the class of chemical combinations are those which have a chemical action on the cement. Whereas these agents fill or help to fill the voids, there is also the advantage of the chemical reaction by which is formed a chemical with waterproofing properties which also assists in the hardening of the concrete.

Summing up the integral method, the main advantage is that it makes possible the use of a wetter mix than ordinary, thereby affording better tamping, etc., producing a denser concrete. It must be remembered that it will not compensate for poor workmanship, poor materials, etc. If the forms allow passage of water, this waterproofing will be washed out, making the concrete weaker than if it had not been used.

5. Membrane Method of Waterproofing.—Membrane waterproofing, while really a surface coating, is now so extensively used that it is listed as a special method. It consists of alternate layers of fabric and tar or asphalt applied either to the exterior or interior surfaces (see Fig. 4). The fabric is usually either felt or burlap impregnated with tar or asphalt. The surfaces are first mopped with hot tar or asphalt and, while that is still soft, covered with the fabric. Usually from three to five layers of fabric are required for a waterproof job. The application of this system is very important as is also the choice of the material; too often it is done by a roofing contractor who knows nothing about waterproofing and who uses ordinary roofing paper instead of fabric.

By the membrane system there is covered up the defects in the concrete due to poor workmanship in leaving porous spots and construction joints. Another advantage is that the fabric is slightly elastic and can, without breaking, bridge structural cracks which are small, preventing leaks at these points.

While this system has these decided advantages, it also has serious disadvantages which are possibly of greater weight. It

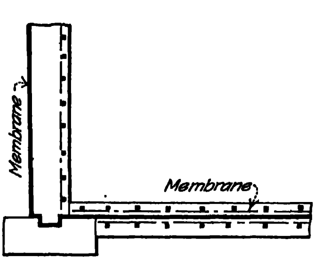


FIG. 4.—Membrane waterproofing method.

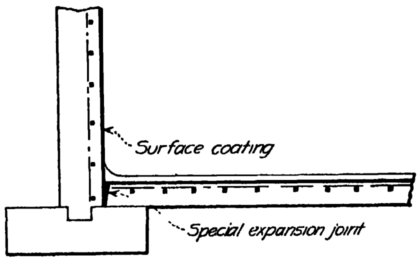


FIG. 5.—Surface waterproofing method.

can not be applied to a wet or cold wall as the tar or asphalt will not hold. If it is applied to the exterior of the walls, extra excavation is required; if applied to the interior of the walls, a special concrete or brick wall is needed to hold the membrane in place and prevent sloughing. It is good practice to place a protecting wall even when the membrane is applied to exterior surfaces. If large or even medium cracks occur, the fabric will not span it and will crack. Expenses involved in repairing leaks in a membrane system are often so high that they are out of the question.

Membrane to secure the best results must be applied while the structure is being built. It should be run over the floor and footings to the exterior of the wall, leaving enough to run part way up the wall. This tends to destroy the bond between the wall and footing and the weight of the wall must be sufficient to prevent slipping.

6. Surface Coating Method of Waterproofing.—The surface coating method (Fig. 5) comprises all materials used to coat the surface after the structure is completed. To secure results with this method all porous spots must be chipped out and replaced with good concrete as it is impossible to waterproof a sponge.

Sodium silicate and magnesium silicate have been used with a certain amount of success. The action is that the silicates act on the free lime in the concrete to form a hard insoluble coating. The life of this coating is limited, requiring periodic treatment.

The Annapolis Mix is a combination of portland cement, coal tar pitch, and kerosene. This has given excellent results when applied to exterior surfaces.

Paraffin has also been used to some extent but its use has been limited because of the high cost. It is applied in the melted state and forced into the concrete with a blow torch, thereby filling the surface pores; a final surface coat is usually applied. This is also limited so far as life is concerned and must be renewed.

The plaster coat method is one system which has been used to a great extent, especially in the East. Briefly this method consists of a dense mortar coat applied either to interior or exterior surfaces. The surfaces are all chipped and roughened so as to secure a mechanical bond for the coating. All bad spots are chipped out and replaced with good dense mortar. Often an integral waterproofing is used in the coating to make it more workable and give a denser mortar.

The main objection to this method is that the plaster obtains the bond with the concrete only because the concrete is roughened. In cases in which it is applied to the interior surfaces where there is much pressure, the coat is liable to be blown off. It has the advantage that it can be easily repaired if leaks develop and will last as long as the structure itself, often prolonging the life of the structure by preventing passage of water through the concrete.

The most used of the surface coatings, especially in the Middle West, is the Ferrous Method. It consists of coating the surfaces with a finely pulverized iron to which has been added an oxidizing agent. These particles are forced into the pores of the concrete by brushing and in oxidizing swell and fill those pores, also forming a bonding coat for additional coats. Each coat must be thoroughly developed before another is applied, if the best results are to be obtained. Usually one or two straight coats of the pulverized iron are applied, followed by alternate coats of slush and iron, the number depending upon conditions. The final coat consists of a cement wash to which has been added a certain percentage of the iron, thereby giving the finished

surface the grey concrete color. If a white finish is wanted, white cement is used.

The objection has been raised that this material becomes only a rust and will not be permanent. It is correct that it forms an iron oxide, of which class rust is one, but that oxide takes the form of limonite which is a combination that is very stable. This system has already stood the test on actual construction for over 14 yr.

On wall work no protecting or restraining wall is needed. The bond is obtained through the swelling action of the iron and is so perfect that on interior surfaces it has stood a pressure of 70 ft. of water. What it will stand on exterior surfaces has never been reached. On floor work it is placed under the topping as a protection against abrasion. This topping becomes an integral part of the floor on account of the bonding action of the iron.

This system is applicable to both exterior and interior surfaces, and is applicable to old as well as to new work. It can be successfully applied to walls that are wet or through which water is flowing. This system does not involve the expense of extra excavation, of a protection wall or of back plastering. Should structural cracks develop, they can be seen and repaired. If desired, paint can be applied directly upon surfaces so treated without any effect on the paint.

The principal objection to this system is that of cost which depends upon the character of the work but, generally, is higher than that of the other systems. This is due to the fact that it must be applied by experienced labor to insure success. It is not a method that can be used by the ordinary layman.

During the past few years there has been a great advance in water-proofing of structures until at present the main problem is to secure a method that will prevent leakage through structural cracks. As yet no such method has been developed so that under the circumstances the best method is that best adapted to the type of construction.

RETAINING WALL FOUNDATIONS

BY JAMES C. MEEM

Although foundations for retaining walls do not differ in essential detail from those of other structures, there are several points which present themselves for special consideration. Ordinarily

the largest horizontal area of a retaining wall is its base, and therefore the design of its foundation is relatively simple. In fact, except for the necessity of a slight depth below the surface of the ground to reinforce the frictional area to prevent sliding, it may be said that the base of the wall itself is all the foundation that is ordinarily required. In view of this necessity for extending the foundation into the soil and for other considerations, the foundation of a retaining wall is usually designed as that of a structure independent of all factors except its own weight and to give a minimum load per area in accord with local conditions and economy.

7. Main Considerations.

(1) Whether or not the pressure of the bank or soil to be restrained by the wall is to be relieved during construction (as in building an independent wall and backfilling), or whether the pressure is to be held with sheeting and bracing during construction.

The placing of the foundation for a retaining wall, while the bank with its pressure load is in position, is not radically different from placing it in a trench except that the braces are inclined props or rakers. If the wall is designed for such operation, the lower tiers of sheeting and bracing may be removed as the wall is built up. Otherwise the sheeting may remain in place and holes may be left in the wall through which the struts or rakers may later be removed.

(2) Whether any action counter to the foundation load should be assumed as coming from the overturning moment against the wall.

As many ordinary retaining walls at some time in their history, for some portion if not all of their length, probably stand alone or nearly so; and further as the ordinary masonry of such walls is not sufficiently unified to render this moment fully effective at the foundation, it is perhaps safer to eliminate this item from consideration—at least as a helpful factor, as noted later, except in specially designed reinforced concrete walls, and in such walls where the pressure may not be expected to be relieved or reduced—as, for instance, clay becoming dry and solidified—and when the economy thereby effected is worth considering.

(3) What action counter to the foundation load should be assumed as coming from the presence (if any) of water under pressure in the soil directly below the foundation.

In the matter of uplift due to the buoyant effort of the water in the soil three typical cases must be considered: (*a*) Foundation resting on piles; (*b*) foundation resting on water-bearing soil; and (*c*) foundation resting on rock.

In the first case where a retaining wall foundation rests entirely on piles, a condition may easily, and probably almost always does, arise in which the soil tends to settle slightly away from the foundation bottom, forming a water pocket over the whole area not actually in contact with the piling. The buoyant effort or uplift acting on the bottom is then

$$P = (A - a)h$$

in which

P = Total pressure in pounds on foundation bottom.
A = Total area in feet of foundation bottom.
a = Total area in feet of piling.
h = Pressure per square foot due to the hydrostatic head of the water.

In the second case the condition is different, as although the foundation may be considered as resting on the equivalent of solid columns of sand, any settlement or subsidence of the soil carries the foundation with it—and therefore a water pocket over a large percentage of the area can not form. Nor can there be, over those areas which are actually supporting the foundation, any water pressure. There may be, however, and probably are, small areas of water pressure corresponding roughly to the voids. In this case the buoyant effort is

$$P = AVh$$

where P = Total pressure in pounds on foundation bottom.
V = Percentage of voids in sand.
h = Pressure per square foot due to hydrostatic head of the water.

Where this factor is a helpful one—that is, favorable to the design or to economy—the designer should be very sure of the value he attaches to V, due to the fact that even coarse soil such as gravel or coarse sand may become clogged with finer soil shutting off practically all water pressure.

With due regard to the safety factor, therefore, it is suggested that, where P tends to exert a helpful pressure, V be made to equal zero, and where the pressure is harmful or against the

design, V be made to equal one-half or 50 per cent. This of course is in the absence of satisfactory tests or positive information as to the true value of V for each special case.

In the last case—foundation resting on rock—the conditions are practically the same as on soil, except that there can be practically no settlement of the support, and the value of V or percentage of voids is normally much smaller and not infrequently is zero.

Where, however, a critical examination of the rock cannot be made and again when P acts against or is a factor harmful to the design, a value of at least 20 or 30 per cent should be assigned to V for safety.

8. Foundations on Piles in Plastic Soil.—A possible condition may arise in which a retaining wall foundation is carried by piles making a firm foundation although their tops are in soil so soft and plastic as to flow under normal pressure—as plastic clay. In this remotely possible case the wall and its foundation are stable while the fill behind it tends to subside and cause the soil around the piles to flow outwardly.

To render such a condition stable, sheet piling should be driven flush with the inside face of the foundation. If the top stringer, or wale, bears against the inner face of the foundation, the piling need not necessarily be driven to firmer soil but only to a sufficient depth so that the pressure on the soil is dissipated laterally.

When the inside face of the foundation cannot be uncovered or exposed, the sheet piling may be driven flush with the outer toe of the wall, bearing against a ranger anchored to the wall foundation by expansion bolts, or reinforced concrete ties or caps, or by any satisfactory anchorage.

It will ordinarily be found in practice, however, that no matter how soft the ground, the bearing piles will act to prevent the flow of soil past them even though the piling is not continuous nor more than normally close.

9. Settlement of Retaining Wall Foundations.—It is usually true that a small amount of settlement is not serious as long as it is not continuous or cumulative. Where this latter is expected or found to exist, piling or the equivalent should be used; but if it is known that the maximum settlement of the soil for the proposed foundation load is not beyond the safe limit set, no further action is necessary and design and construction may be proceeded with.

DAM FOUNDATIONS

By Charles H. Paul

10. Main Requirements.—The main requirements to be considered for a dam foundation are: bearing power, water tightness or control of seepage; prevention or control of upward pressure, prevention of sliding of the dam on its foundation or in the foundation itself, and protection against scour below the downstream toe or apron.

11 Design Depends on Kind of Foundation Material.—The type of the dam to be built, and its design, depends on the character of foundation available. Inasmuch as dams are being built successfully on such material as light river silt, it is evident that almost any material, however unfavorable, may be used for a dam foundation, provided it is properly prepared, and provided further, that a suitable design is worked out to fit the foundation conditions. In other words, some type of dam may be built on almost any foundation. On the other hand, failure to understand foundation conditions, or to appreciate their importance, has often resulted in disaster, as at Austin, Texas, or in greatly increased cost and delay, as at Hales Bar, Tenn.

12. Meaning of the Term "Foundation."—It must be remembered that the entire area upon which the dam rests is included in the term "foundation." It is not sufficient to examine only the lower portion of the foundation, but the examination should be continued all the way to the ends of the dam, and out into the abutments, to make sure that the higher portions of the foundations, and the abutments, also meet the requirements. There are cases on record where this precaution has been neglected, with unfortunate results, as at the Cedar Lake dam at Seattle.

13. Foundation Material.—*Solid rock* is the only suitable foundation for a high masonry dam, say about 200 ft. or higher. By solid rock is meant firm hard rock, without open seams, fissures, or faulting. Obviously such rock is also suitable for dams of any other type. Its bearing power is sufficient, and the requirements as to water tightness, upward pressure, sliding, and scour, are more easily controlled than with any other foundation material. Even with solid rock, however, surface indications must not be relied upon entirely, and the importance of subsurface examination, and thorough preparation of foundation, must not be overlooked. These subjects are discussed in Arts. **14** and **15**.

Softer rock, or *rock of poorer quality*, if not too badly, fissured and *hard shale*, are suitable foundation material for masonry dams of moderate height, say less than 150 to 200 ft. as well as for reinforced concrete, rock fill, earth, or timber dams. Although any ordinary rock, and the harder shales, are usually sufficient as to bearing power, the other requirements are not so easily satisfied, particularly as to water tightness, upward pressure and scour, in the case of the softer rock, and as to sliding and scour in the case of shale. *Cavernous rock*, for any dam foundation, should be avoided if possible, as it is extremely unreliable, even after the most careful grouting or other treatment. The difficulties encountered at Hales Bar dam (Tenn.) illustrate this. Sub-surface examinations become increasingly important as the character of the material is less reliable, and special treatment is often necessary to meet the various foundation requirements. Many of the softer rock and shales deteriorate rapidly upon exposure, and that feature must be kept in mind not only in the design of the structure, but also in preparing the foundation to receive it.

Clay is usually regarded with suspicion for a dam foundation, although with proper treatment the harder clays may be suitable for most types of dams, except masonry dams of considerable height. The clays vary so much in character, and are so affected by changes in moisture conditions, that great care must be used in determining their suitability in any case. By going to greater depth, or confining the material by curtain walls or sheet piling, or by loading the material outside of the structure itself, or by drainage, a clay foundation which otherwise would be unsatisfactory may be made to meet the requirements. Piling, if practicable, may be used to increase the bearing power, or to take most or all of the load. Given the necessary bearing power, the requirements as to sliding and scour are the ones to consider most carefully. Clay foundations must always be protected from overflow, or discharge from outlets or spillways, until velocities have been reduced to the point where scour or wash will cause no damage.

Gravel and sand are commonly used as foundations for low masonry dams, say 50 ft. or less in height, and for other types of dams of any reasonable height. Such foundations must be studied carefully, however, inasmuch as few if any of the requirements are satisfied without special treatment of the foundation,

or special design of the structure to meet the conditions. Bearing power is usually sufficient, except for masonry dams, where piling or a widened base may be necessary. Seepage may be controlled by cut-offs, core wall, or an impervious upstream blanket. Upward pressure must be provided for in the design. Scour may be prevented by a timber or rock apron.

Sand and silt have been used successfully in many cases as foundation material for earth dams, and for low dams of most any other type, including reinforced concrete dams of special design. All the precautions mentioned in the preceding paragraph must be considered even more carefully in the case of these lighter materials, and in addition, it is usually necessary to make some modifications in design to meet the unfavorable foundation conditions. The Gatun dam (Panama) and the Laguna dam (Arizona) are cases in point.

Generally speaking, any dam is an important structure, and the selection of type and design to suit the foundation available should be based on sound judgment and experience, with questions of doubt always decided on the conservative side. In the case of any dam of considerable size, the best advice that can be obtained should be secured. Failure to observe good engineering practice accounts for practically all the failures of dams that have ever occurred. A conspicuous example of this, as applied particularly to foundation conditions, was the failure of the Austin dam (Texas).

14. Examination of Foundations.—For dams of importance it is desirable, whenever possible, to have a careful geological examination of the foundation conditions. An investigation and report by an expert practical geologist is very much worth while in most cases. Such reports should deal with the geological characteristics of the rock, its reliability as to bearing power, deterioration, etc., as well as probability of fissures and faulting, or former upheavals or disturbance. Even slight settlement is not permissible in foundations for a masonry dam, except possibly for low dams specially designed with that in view. For earth or rock fill dams, however, slight settlement of foundation during construction may not be objectionable, provided there is no danger of further settlement after construction is finished. But unequal settlement near outlets or spillways due to unequal loading, must be guarded against by special design.

Where bearing power is questionable, field test may be made by clearing off a small space and loading it by means of a platform or box supported on a base of known area and loaded with sand, pig iron, rails, etc., of known weight. Unfortunately such tests, for practical reasons, must be limited to an area of only a few square feet at most, and the results are difficult to interpret correctly, as a small area will carry a greater load for a short time, than a larger area will carry for a long period; therefore, long period tests should be made if possible, and a large factor of safety is necessary to allow for inaccuracy of results. Values for supporting power of soils are given in table on pp. 22 and 23.

Wash borings are most commonly used to examine foundations, other than rock or hard shale. Although they usually give fair information, they are not thoroughly satisfactory, as it is extremely difficult to obtain representative samples, even by the so-called dry sampling method, which should always be used when possible. If at all feasible, wash borings should always be supplemented by at least a few test pits to check up the information obtained from the borings. Wash borings, followed by core drilling, will usually give fairly accurate information as to the surface of bed rock, provided they are spaced closely enough. Wash borings alone, however, may stop on boulders, and thus be misleading.

Core drilling should be resorted to for examination of rock foundation. The holes should be carried into the rock to a depth of 20 ft. or more, and a careful record should be kept of the amount of core obtained. A few holes should be put down to a considerably greater depth, in order to make sure that there are no faults in the foundation below the rock surface. At the Shoshone dam (Wyoming) and also at the Arrowrock dam (Idaho) some of the core drill holes passed through an overhanging shelf of rock, where the side of the cliff had once been undercut and afterwards filled in by the deposition of gravel. The final surface of the rock was found at a greater depth than was at first indicated. Had the core drillings been confined to shallow holes, the result in these cases would have been misleading. It is not uncommon to encounter immense boulders lying on top of the bed rock, and here again core drilling to a considerable depth is necessary in order to make sure of the foundation conditions.

Open test pits afford the only opportunity to inspect the material in place. Even though they are comparatively expensive, a few of them, at least, are very desirable, especially in the

case of important structures, or where foundation conditions are at all questionable.

15. Preparation of Foundation.—The preparation of the rock foundation for a high masonry dam is one of the vital features of construction. All loose or soft rock should be carefully cleaned off and removed, and the surface scrubbed clean with water and stiff brooms or wire brushes. This scrubbing should be continued until the foundation is absolutely clean, even to the extent of

FIG. 6.—Cleaning up bed rock foundation for the highest dam in the world (Arrowrock, Idaho).

sopping up and disposing of the dirty water with sponges and buckets. The foundation should then be slushed with a thin neat cement grout, scrubbed in with brooms, just in advance of the placing of the concrete or masonry. This tends to take up any loose particles and incorporate them in the masonry. A layer of rich mortar, say 1:2, an inch or more in thickness, should be applied and kept just ahead of the concrete or masonry construction. If the dam is of concrete, it is good practice to make the first pourings in as thick layers as is practicable, say 6 or 8 ft., so that the weight of the fresh concrete will help to make a good bond with the foundation.

Often there are seams of softer material running through a foundation of hard rock. Such seams should be excavated or cleaned out to sufficient depth and filled with concrete or grout. At the Arrowrock dam (Idaho) a seam of porphyry running diagonally across the foundation was cut off by a shaft 40 ft. deep, sunk down the porphyry seam and filled with concrete which was keyed into the granite on either side of the shaft.

Fig. 7.—A section of rock foundation for Arrowrock dam, Idaho, cleaned off ready for placing of concrete.

Blasting near neat lines must be done with great care, so as not to disturb or loosen up the rock. Short holes and very light charges should be used, where such blasting is necessary in trimming up a foundation, and for the final trimming, use of bars or picks or wedges is much to be preferred. Where heavy excavation is necessary in material requiring blasting, this precaution should be kept in mind in drilling and shooting the primary holes, especially in seamy material where the effect of a blast may be felt at some distance from the hole.

Grouting of rock foundations for masonry dams is often resorted to as a matter of precaution. In fact, it is standard practice in the construction of high masonry dams, as at the Arrowrock dam (Idaho), Elephant Butte dam (New Mexico) and at the Kensico

dam (New York), and it is often required to seal up an otherwise questionable foundation, as at the Estacada dam (Oregon).[1] It is a development of the last 15 or 20 years, and therefore no mention of it will be found in early records of dam construction.

Grouting, in order to be effective, must follow a carefully prepared plan involving the use of test holes, as the work pro-

Fig. 8.—Cleaning up and grouting rock foundation for the outlet and spillway structure of the Lockington dam, Ohio.

ceeds, to show whether the desired results are being accomplished. The grout holes are usually drilled in two or more parallel lines, with the test holes between. Each grout hole should be drilled and grouted before another hole is drilled, otherwise the grout is likely to escape through the open holes, instead of being forced into the seams. All holes should be thoroughly washed out and tested for leakage under water pressure before the work starts. Neat cement grout should be used, unless the seams in the rock are open and take the grout very freely. In that case, some sand may be added until the hole begins to tighten up. Grout is usually mixed in proportions 1 cement and 3 water, to 1 cement and 5 water, depending on the tightness of the holes. The idea, of course, is not to seal the hole as quickly as possible, but on the contrary, to force in as much grout as the foundation will take. When the grouting of a hole has started, the flow of grout should be continuous, until it will take no more. Grouting machines with two tanks, or a combination of two machines properly connected to the discharge line, are used to accomplish this. The grout is driven home by air pressure or water pres-

[1] See *Transactions Am. Soc. C. E.*, vol. 78, 1915.

sure, depending on the type of machine used. Usually it is best to begin with a low pressure and work up gradually to the maximum. The maximum pressure to be used must depend on local conditions. In rock with horizontal seams care must be taken not to lift the rock. It is good practice to bring the grout holes up through the first layer of masonry and do the grouting after the foundation is covered. This also tends to grout the joint between the dam and its foundation. Grouting as a precautionary measure is excellent treatment for the foundation of a dam of any considerable size, but at best the results are uncertain, and where the safety of the structure is contingent on the success of grouting operations, it is better to look elsewhere for a foundation. In other words, grouting as a precaution is to be recommended. Where absolutely necessary for the safety of a proposed dam, it is a doubtful expedient.

Grouting of sand and gravel has been attempted at various times, but no satisfactory method has yet been worked out,—for dam foundations at any rate. In the light of present knowledge it should not be considered for such purpose. What is usually accomplished is simply to get a small mass of grouted material just at the end of the pipe.

To key the dam into its foundation, it is customary to excavate a cut off trench or keyway across the foundation along the heel or upstream face of the dam. The depth of this keyway will depend on the character of the rock. It should be at least 3 or 4 ft. deep in any case, and should be continued up the abutments, and along the full length of the dam. It may be as narrow as is economical to excavate. Its sides should be nearly vertical, with clean square corners. Wherever practicable, the use of a channeling machine is recommended. In some cases one or more additional keyways may be desirable, in which case they should be parallel to the first and located within the upstream third of the dam. If the rock is bedded in horizontal layers, the matter of keyways or cut-offs is of great importance, and they should be carried to sufficient depth to cut off any seams which might permit seepage or sliding.

It is sometimes thought desirable to scarify the rock foundation of a masonry dam. This is seldom necessary, as the work of cleaning off the foundation usually leaves it sufficiently rough to furnish a good bond and prevent sliding. Even when the rock is clean and water worn, it is seldom so smooth and level that

sliding is possible without actually shearing the masonry or the foundation itself.

The foregoing discussion of the preparations of foundation refers particularly to solid rock foundations for masonry dams. *For the softer rocks, shale or clay* similar procedure should be followed, with such modifications as may be suggested by the character of the foundation material or the size or importance of the structure. Clay, shale, or rock which might decompose rapidly when exposed, should not be excavated to neat lines when the bulk of the excavation is being removed. It is best to excavate such material only to within a few inches, or perhaps a foot or more, of final depth, and then trim to neat lines just ahead of the placing of the concrete or the masonry, so that the foundation may be covered as soon as possible after being exposed. Anchoring the dam to its foundation is sometimes resorted to in case of rock with horizontal clay seams, or shale, to guard against sliding or the effect of upward pressure.

The purpose of this is to make the foundation material, for a certain depth, an integral part of the dam structure. It is usually accomplished by drilling holes, properly spaced, to the depth required, and grouting in reinforcing bars which will be carried up and tied in to the masonry in such a manner as to effectually tie dam and foundation together. Or, if protection against sliding only is required, these bars may project into the masonry only far enough to act as dowels. Shale and clay foundations should not be scrubbed with water or grout, for obvious reasons, but should be treated with a layer of rich mortar as the masonry is being placed. In such foundations, too, it is necessary to go to sufficient depth to insure against sliding or the softening of the foundation by the action of water. Deep cut-offs or curtain walls of concrete, are also desirable at both upstream and downstream faces of the dam. A well designed drainage system may be required at the downstream toe.

Overflow dams on the softer foundations, must be protected downstream by aprons of concrete, rock, or timber, carried to sufficient distance to prevent any possible erosion that would endanger the structure. The design of a suitable apron in such cases is fully as important as the design of the dam itself. Aprons should be so designed that the standing wave will always occur well back on the apron, and should be protected at the downstream end by heavy rock, paving, or riprap. Discharges from

outlets and spillways may be handled in a similar manner. Hydraulic jump pools, as used in the Miami Conservancy dams (Ohio), are very satisfactory for reducing the velocities and thus preventing scour. For vertical drops, water cushions are effective, provided they can be given a depth of 20 to 40 per cent of the drop. Even hard rock is not proof against the destructive action of falling water, the water in the seams acting as a wedge taking the impact of the falling water and eventually rupturing the rock. A water cushion or a heavy concrete apron will prevent this trouble.

A clay foundation which is questionable as to bearing power, may be improved by several of the expedients suggested heretofore, namely: spreading foundation, deeper excavation, use of piling, or confining by cut-offs or curtain walls. A soft clay may sometimes be greatly improved by driving short piles fairly close together. Such piles, however, must be below permanent water level, or otherwise protected against rotting. Loading the foundation outside of the structure proper, may also be successful. At one of the embankments forming the Deer Flat reservoir (Idaho), the downstream toe of the embankment was extended by means of a heavy blanket of gravel, which served the purpose of loading the partially saturated clayey material just outside the embankment foundation, which otherwise might have squeezed out and allowed settlement. This blanket served another useful purpose of providing drainage for any seepage which might find its way through the foundation material under the embankment.

Prevention or control of upward pressure in masonry dams is a subject which has been under lively discussion for many years.[1] While there is wide divergence of opinion as to how such pressure should be computed, all are agreed that it must not be disregarded and should be reasonably provided for. As a matter of fact, its prevention or control is not difficult usually. In most masonry dams it is feasible to construct drainage galleries lengthwise of the dam, close to the upstream face, and high enough above downstream water surface to afford drainage to the downstream face. A line of open drainage holes or weep holes may be drilled into the foundation and carried up to the drainage gallery to provide relief for any upward pressure which may exist. This has been done at many of the important dams built in

[1] See *Transactions Am. Soc. C. E.*, vol. 75, 1912.

recent years, including the Elephant Butte dam (New Mexico), Arrowrock dam (Idaho), and several others. The drainage galleries should be located near the upstream face so that the weep holes may be placed just downstream from the upstream cut-off, or the grout holes if any. In a grouted foundation the weep holes should not be driven, of course, until the grouting has been completed.

Cut-off and curtain walls in the upstream third of the foundation, impervious upstream aprons, and drainage of the downstream toe, all tend to prevent the action of upward pressure. Arching the dam in plan, or even building it on only a slight curve, will also help to take care of the uplift stresses. Any

Fig. 9.—A portion of the foundation for the Englewood hydraulic fill dam, Ohio, cleaned up ready for excavation of cut-off trench.

treatment of the foundation which leaves the upstream third more impervious than the downstream, will be effective in reducing or preventing upward pressure. But the combination of weep holes and drainage galleries (which will also serve as inspection galleries) is the simple and positive method to be used whenever practicable.

Gravel and sand, or sand and silt foundations for earth or rock fill dams should be stripped of all light loamy top soil, peat, vegetable matter, or other perishable matter, and roots more than an inch or so in diameter should be grubbed out. One or more cut-off trenches, or core trenches, of reasonable depth, should be excavated across the valley and up the abutments, to be filled with impervious material. This should be done for exploration purposes, even if not required to prevent seepage.

Springs encountered in the foundations should be controlled by diversion or by proper drainage. Bearing power of such foundations is usually sufficient for earth or rock fill dams of reasonable height, because of the relatively wide base required, and, at any rate, slight settlement during construction of such dams does not matter. The importance of loss of water will determine to what extent measures for prevention of seepage must be carried

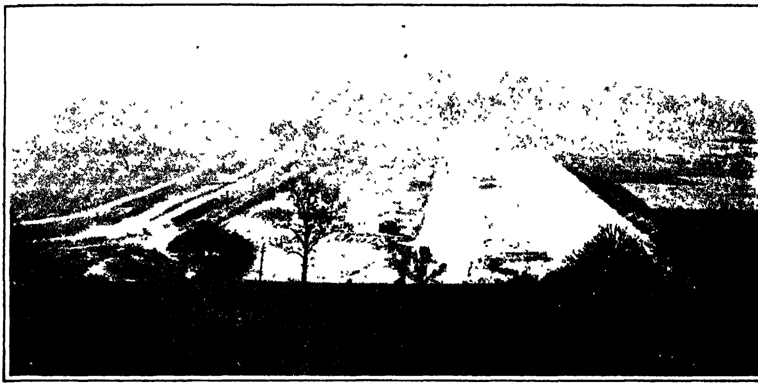


FIG. 10.—The foundation for the Huffman dam, Ohio, cleaned off, and cut-off trench excavated, ready for beginning of hydraulic fill.

out. Where even slight loss of water is objectionable from an economic standpoint, the core trench must be carried to rock or impervious material, or, if the depth is too great for proper handing of a core trench, a concrete or sheet pile cut-off may be substituted. If neither of these is practicable, the next best thing is a blanket of impervious material connecting with the impervious section of the dam and extending for some distance upstream.

Even where loss of water is not objectionable, seepage must always be controlled to the extent that flow through the foundation material will not have enough velocity to wash out the finer particles. Lengthening the line of seepage travel by means of cut-offs, or by an impervious upstream blanket, will accomplish the result desired. A conservative rule, in ordinary porous sand and gravel, is that the ratio of length of travel to head, shall be not less than about 8 or 9 :1. It often happens in wide river valleys, that there is an overburden of impervious material on top of the porous sand and gravel. In such cases, the thickness of this impervious layer should be determined by means of a

post hole digger, or other suitable apparatus, at various points upstream from the impervious portion of the dam. The thin spots, if any, may then be reinforced by artificial "patches" of rolled material, until the thickness of the impervious blanket at any point will be not less than 3 to 6 ft., depending on the head of water to which it will be subjected.

The foundations of the five hydraulic fill dams of the Miami Conservancy District (Ohio) were all prepared in this way. Protection against scour from discharge of outlets and spillways may be secured by means of riprap, paving, aprons, water cushions, or hydraulic jump pools, as mentioned heretofore. Whenever practicable, outlets and spillways should be located on the same side of the river, to save expense in protecting against scour. Strange to say, this is a detail that is sometimes overlooked.

Foundations of gravel and sand for low masonry dams should be prepared in the same manner as for earth or rock fill dams, special attention being given to the requirements as to bearing power, sliding, and upward pressure. As masonry dams on such foundations are usually of the overflow type, the design of the downstream apron is especially important, and too much care cannot be taken in protection against scour at that point. All that has been said heretofore concerning aprons, paving, riprap, etc. should be called to mind in this connection. Here again the best of judgment is required to obtain satisfactory results.

The use of *piling*, to meet the requirements as to bearing power, is often necessary, and in such cases the piling should be designed to carry most, if not all, of the load. A sheet pile cut-off at the upstream edge of the dam is usually desirable, and at the downstream toe a row of round piles driven close together, or of sheet piles with openings to permit drainage, will act to retard undercutting in case the apron is not fully effective. The impervious upstream apron is also desirable if practicable. Upward pressure must be reckoned with to an extent dependent on the porosity of the foundation material. As a matter of fact, in cases of this sort, the preparation of foundation and the design of the structure are so interdependent that it is hardly possible, here, to do more than suggest general precautions to be observed.

A word ought to be said about *pump sumps*. It is important that sumps be set low enough, before the final cleaning up of the foundation is attempted, so that the water may be kept out of

the way during that process, and also while the first courses of masonry are being laid. Very often the location of a pump sump may mean the difference between a good foundation job and a poor one. It is a good paying investment, on any wet foundation work, to give special attention to location of sumps, so that the water level in the pit may always be under proper control, within the limits required.

No two foundations are alike in all respects. Rocks of different kinds, and of different quality, as well as various combinations of gravel, sand, clay, and silt, each require special study and proper treatment. In some cases two or more entirely different materials will be present in the same foundation. An attempt has been made to discuss a few typical cases, leaving it to the judgment of the engineer to work out the best solution for his own particular problem.

MACHINERY FOUNDATIONS

By S. E. Slocum

16. Kinetic Reactions of Machinery.—Foundations are subjected to two general classes of loading, static and kinetic. A static load is one which does not vary with the time, such, for example, as the dead weight of a building or machine. In structural design, live loads such as the weight of a train on a bridge, the weight of merchandise in a store house, or of an assembly of people in a building—are also classified under this head. The characteristic feature of static loading is that for a given structure the load carried by the foundation at any given time is constant in magnitude and direction. Consequently for static loading, the main purpose of the foundation is to provide adequate resistance to the force of gravity.

In designing foundations to carry static loads, two general principles are followed. These are:

1. Provision of sufficient bearing area to prevent permanent settlement or deformation of the supports.

2. Equality in distribution of pressure.

The application of these two principles is fully treated in other parts of this volume, and will not be considered further in this chapter.

In machinery, the static loads due to the dead weights of the machine and its attachments are usually of minor importance

as compared with the kinetic reactions produced by the motion of the various moving parts. Each moving part when accelerated or retarded gives rise to inertia forces in accordance with Newton's law

$$\text{force} = \text{mass} \times \text{acceleration}$$

In general, such forces are periodic and give rise to vibration. When there are several moving parts, these in general set up a corresponding number of separate vibrations in the machine, so that the resultant effect is usually very complex. The problem of designing a foundation to take care of the inertia forces set free by machinery in motion must therefore be solved by entirely different methods from those used in designing foundations for static loads.

The requirements which any particular foundation must satisfy depend largely on the kind of machine it is intended to support. For example, in machine tools the principal requirement is rigidity, in order to maintain accuracy of operation. Machines of this type are therefore usually of rugged construction with massive foundations.

At the opposite extreme are such machines as aircraft, in which the motor has practically no foundation so far as its mass is concerned. This requires an absolutely different type of mounting, the proper design for the support in this case being such as to minimize dynamically the effects of vibration so far as this is possible, and to absorb the residual energy of vibration by some form of damping.

In designing the foundation for electrical machinery, and in fact for all types of modern high speed machinery, the elimination of vibration is an important factor. This can be partially accomplished in the machine itself, but there is always an appreciable amount of residual vibration which must be taken care of by the foundation.

When the kinetic reactions caused by operating a machine do not occur with such frequency as to produce vibration, they are frequently of such amount and nature as to cause excessive shock and noise, often to such an extent as to endanger the building by the jar, or produce nervous fatigue in the operatives by the noise. A familiar example of this kind of machine is a battery of looms in a textile factory. Usually such machines are mounted directly on the floor, the result being that it is necessary to house them in

specially reinforced buildings, and also making it necessary to relieve the operatives at frequent intervals. With properly designed foundations, it would be possible to relieve such conditions very materially.

The three main principles underlying the design of foundations for machinery are therefore.

1. To overcome dynamically the effects of free inertia forces and couples by balancing such kinetic reactions within the foundation itself so as to cause them wholly or partially to cancel.
2. To prevent synchronism with adjoining machines or structures, by proper location or distribution of loads, by correct proportioning of structural members, or by insulation.
3. To absorb residual vibration by means of dampers incorporated in the foundation.

17. Weight Required in Massive Foundations.—The use of heavy foundations under machinery is the simplest and most primitive means of providing resistance to the kinetic reactions arising from the moving parts of the machine. In many cases as, for example, in marine installations, power plants for aircraft or automobiles, and factories located on the upper floors of buildings, the use of a massive foundation is out of the question, and more scientific means must be resorted to for preventing vibration, or damping the effect of unbalanced inertia forces. Where a massive foundation can be used, however, it is effective as an inertia damper, although never efficient from the standpoint of power losses and strain on the machine.

There is no standard practice in designing heavy foundations for machinery, and so far as the writer is aware, no general method for designing such foundations has ever been given. Each designer is a law to himself and lays out his foundation by eye, usually with the simple requirement of making it fit in the space available. It is evidently unscientific as well as uneconomical to mount a highly fabricated machine on a foundation which is largely guess work, and the following method of analysis may serve as a basis for determining the weight required in a massive foundation.

Consider a single cylinder horizontal engine, say of the Corliss type (Fig. 11). As in the case of all internal pressure engines, the steam pressure in the cylinder produces equal and opposite forces on the piston and cylinder head. Consequently these forces cancel out as regards the machine as a whole, and therefore have no effect on the foundation. The moving parts, how-

ever, possess mass and hence inertia, and when accelerated or retarded, give rise to inertia forces or kinetic reactions, in accordance with the fundamental law *force* = *mass* × *acceleration*. The resultant of all these inertia forces at any given instant is, then, the free force transmitted to the foundation at that instant. These inertia forces are in general harmonic, or approximately so, and therefore periodic, and consequently their resultant is also periodic.

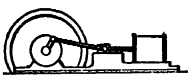


Fig. 11.

In the present case, in order to express the required relations mathematically, let

W_1 = weight of eccentric rotating parts, including crank pin, crank cheeks, and crank end of connecting rod.
W_2 = weight of reciprocating parts, including piston head, wrist pin, piston rod, cross head and reciprocating end of connecting rod.
W_3 = total weight of engine.
W_4 = weight required in foundation.
l = length of connecting rod between centers of pins.
r = crank radius.
$q = \frac{l}{r}$.
n = speed of engine in revolutions per minute.
ω = angular velocity of crank = $\frac{2\pi n}{60}$.

For uniform speed of revolution the rotating parts have a constant central acceleration of amount $r\omega^2$ and hence their inertia produces a centrifugal force of amount

$$C = \frac{W_1}{g} r\omega^2$$

directed radially outward from the center of rotation.

The maximum acceleration of the reciprocating parts occurs at the ends of the stroke where the direction of motion is reversed. If the connecting rod was infinitely long, the motion of the reciprocating parts would be harmonic, and their maximum acceleration would be $r\omega^2$. For a connecting rod of finite length, however, the acceleration depends on the ratio of the length of the connecting rod to the crank radius, namely on $q = \frac{l}{r}$. In this case it is easily shown that at the out-end of the stroke the

acceleration is $r\omega^2\left(1 - \frac{1}{q}\right)$, whereas at the in-end of the stroke it is $r\omega^2\left(1 + \frac{1}{q}\right)$. The inertia force due to this acceleration or retardation is

$$F = \frac{W_2}{g} r\omega^2 \left(1 \pm \frac{1}{q}\right).$$

Consequently the maximum kinetic reaction exerted on the shaft is

$$\frac{W_1}{g} r\omega^2 + \frac{W_2}{g} r\omega^2 \left(1 + \frac{1}{q}\right)$$

This periodic inertia force is resisted jointly by the inertia of the masses to which it is transmitted, including the mass of the machine, the mass of the foundation, and the mass of that part of the soil or subfoundation which may be assumed to act as a unit with it. In other words, the practice of rigidly anchoring a machine to a massive foundation is based on the principle of using the relatively small accelerations set up in a large mass, namely foundation and underpinning, to balance the large accelerations of relatively small masses—namely, the moving parts of the machine. For this reason the soil on which a foundation rests usually adds greatly to its effectiveness, for the mass of the soil is also accelerated just so far as the disturbance transmitted to it by the foundation extends.

Let k denote the ratio of the mass of soil accelerated to the mass of the foundation proper, and let a denote the average acceleration for the entire mass set in motion. Then the condition for equilibrium against horizontal translation in the present case is

$$\frac{W_3 + W_4 + kW_4}{g}(a) = \frac{W_1}{g} r\omega^2 + \frac{W_2}{g} r\omega^2\left(1 + \frac{1}{q}\right).$$

Since the motion of the foundation is periodic with the same period as the engine, it is a sufficiently close approximation to assume that it is harmonic. If the amplitude of this harmonic motion is $2b$, then $a = b\omega^2$. Substituting this value, cancelling out the common term $\frac{\omega^2}{g}$, and solving for the required weight of foundation W_4, we have

$$W_4 = \frac{r}{b(1 + k)}\left[W_1 + W_2\left(1 + \frac{1}{q}\right)\right] - \frac{W_3}{1 + k}$$

The horizontal reaction applied to the bed plate of the engine is of course accompanied by a vertical overturning couple acting on the foundation in the plane of the motion. In the present case, however, the effect of this couple is unimportant in comparison with the lateral motion due to the horizontal reaction.

Illustrative Problem.—In an engine of the Corliss type assume the following approximate weights and dimensions:

Weight of rotating parts = 150 lb.
Weight of reciprocating parts = 400 lb.
Total weight of engine = 12 tons.
Speed = 120 rev. per min.
Length of connecting rod = 5 ft.
Length of stroke = 20 in.

Find required weight of foundation to limit its lateral motion to 0.005 in. in either direction from a position of rest.

In the above formula, k is an empirical factor, the value of which depends on the nature of the soil or sub-foundation. In the present case assume $k = 10$ as an average value. Substituting the given numerical values in the above formula, the result is

$$W_4 = 55 \text{ tons}$$

as the actual weight required in the foundation under the assumed conditions to limit its motion to the amount specified.

18. Design of Beam Supports.—As mentioned in Art. **16**, in designing supports for machinery, their dimensions cannot be determined from the strength required to support the dead weight of the machine. Evidently the supports must be designed to offer adequate resistance to the kinetic reactions developed by the moving parts of the machine, but this requirement usually resolves itself into the condition that the supports shall be so designed as to prevent any possibility of synchronizing with the operating speed of the machine. In other words, foundations for machinery can not be regarded as safe or efficient unless they are reasonably free from vibration.

When a machine is supported on an elastic framework of any kind, the motion of the machine in general will produce vibration of the support. Every elastic framework, however, possesses a certain definite natural frequency of vibration, and when the speed of the machine reaches the same numerical value as this natural frequency of the support, the vibrations of the latter become excessive. This is the so-called *critical speed* for the given construction. The existence of vibration in a structure is never in itself a sign of weakness. In any structure, vibrations are sure to arise whenever its natural frequency synchronizes

with the frequency of the exciting force, while a weaker structure under the same conditions would not vibrate to the same extent because of lack of synchronism. In designing supports for machinery it is therefore always essential to so proportion them as to make certain that the running speed of the machine never approaches very closely to the natural frequency of the support.

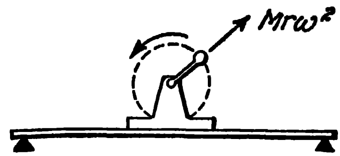


FIG. 12.

To explain the method of dimensioning beams to avoid synchronism, suppose that a machine of weight W rests at the center of a simple beam of length l (Fig. 12). As a first approximation neglect the weight of the beam itself in comparison with the load. Then the maximum deflection d of the beam occurs at the center and is of amount

$$d = \frac{Wl^3}{48EI}$$

where E is the modulus of elasticity for the material of the beam, and I is the static moment of inertia of its cross-section. The natural period P of free vibration of the beam is then

$$P = 2\pi\sqrt{\frac{d}{g}}$$

where g denotes the acceleration due to gravity. Inserting in this the value of the deflection d, we find

$$P = 2\pi\sqrt{\frac{Wl^3}{48EIg}}$$

Let n denote the speed of the machine in revolutions per minute. Then since the period P is expressed in seconds, we have

$$n = \frac{60}{P}$$

Consequently the lowest critical speed for the machine—that is, the speed at which the amplitude of vibration will be a maximum is

$$n_{\text{critical}} = \frac{30}{\pi}\sqrt{\frac{48EIg}{Wl^3}} = 1{,}300\sqrt{\frac{EI}{Wl^3}}$$

E, I, and l being expressed in inch units, W in pounds weight, and n in revolutions per minute. If the actual running speed of the machine is likely to approach this value, it will be necessary to change the dimensions of the beam. Strengthening the beam

by increasing its moment of inertia I raises its natural frequency, whereas weakening the beam by decreasing I lowers its natural frequency.

If the operating speed was considerably higher than the natural frequency of the beam, there might be little vibration at this speed, but in starting and stopping the machine, it would always have to pass through this critical speed, which might prove dangerous. It is therefore desirable that the operating speed should lie well below the lowest critical frequency of the support.

If it is desired to include the weight of the beam in the formula for critical speed, this may be done approximately as follows. The load carried by the beam in this case is the weight W of the machine and the weight wl of the beam itself, where w denotes its weight per unit of length. The deflection at the center therefore consists of two parts. That due to the concentrated load W which is

$$d_1 = \frac{Wl^3}{48EI}$$

and that due to the uniformly distributed load wl which is

$$d_2 = \frac{5wl^4}{384EI}$$

Therefore the total deflection at the center is

$$d = d_1 + d_2 = \frac{l^3}{48EI}(W + \tfrac{5}{8}wl)$$

which is the same as for a single concentrated load of amount $W + \frac{5}{8}wl$. Assuming that the natural frequency of vibrations of the beam is the same for a single concentrated load as when part of the load is uniformly distributed, the formula for critical speed becomes in this case

$$n_{\text{critical}} = \frac{30}{\pi}\sqrt{\frac{48EIg}{(W + \frac{5}{8}wl)l^3}}$$

This analysis is only approximate and the fraction $\frac{5}{8}$ is therefore not exact. A rigorous analysis shows that the proportion of the weight of the beam which should be added to W is $\frac{17}{35}$ instead of $\frac{5}{8}$.[1] Therefore the required formula for practical use becomes

$$n_{\text{critical}} = \frac{30}{\pi}\sqrt{\frac{48EIg}{(W + \frac{17}{35}wl)l^3}} = 1{,}300\sqrt{\frac{EI}{(W + \frac{17}{35}wl)l^3}}$$

[1] The correct analysis of this problem has been given by S. Timoshenko in Russian.

To illustrate how this formula is used, suppose we assume as our working condition that the lowest critical frequency for the beam shall be ten times as great as the actual speed of the machine or

$$n_{\text{critical}} = 10n$$

and determine the size of the beam accordingly. As a first approximation neglect the weight of the beam. Then

$$n_{\text{critical}} = 10n = \frac{30}{\pi}\sqrt{\frac{48EIg}{Wl^3}}$$

whence

$$I = \frac{n^2Wl^3}{1{,}690E}$$

Using this value of I, the dimensions of the beam may be determined and its weight calculated, and as a check its critical speed may be redetermined, using $W + {}^{17}\!/_{35}wl$ for the concentrated central load.

The above considerations also indicate that it is desirable to distribute the load over the beam non-uniformly, as in general this has the effect of raising the critical frequency of the beam.

19. Design of Column Supports.—When a machine is supported on columns, as for example when machinery is installed on the upper floors of a building, it is of course just as essential as in the case of beams that the operating speed of the machine shall not synchronize with the natural frequency of vibration of the columns.

To explain how synchronism may be avoided in this case, consider a weight W supported on a column of length l, and as a first assumption neglect the weight of the column itself in comparison with W (Fig. 13). Under the action of a horizontal force applied to the top of the column, it will deflect like a cantilever beam. This horizontal force is made up of the horizontal components of the kinetic reactions produced by the motion of the machine. Denoting the horizontal force acting on one column by H, the lateral deflection of the upper end of the column will be

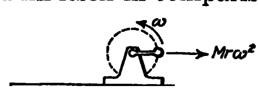


Fig. 13.

$$d = \frac{Hl^3}{3EI}$$

where I denotes the moment of inertia of a cross section of the column. Therefore d is proportional to H which is a sufficient

condition that the lateral motion of the column shall be harmonic. The fundamental equation for simple harmonic motion is

$$m\frac{d^2x}{dt^2} = -kx$$

where k denotes the value of the disturbing force at unit distance from the position of rest. In the present case k is the value of H for $d = 1$; consequently

$$k = \frac{3EI}{l^3}$$

The period of the harmonic motion is

$$P = 2\pi\sqrt{\frac{m}{k}}$$

where m denotes the mass of the body, and the frequency f is

$$f = \frac{1}{P} = \frac{1}{2\pi}\sqrt{\frac{k}{m}} = \frac{1}{2\pi}\sqrt{\frac{3EIg}{Wl^3}}.$$

The critical speed for the structure, namely the operating speed of the machine which synchronizes with the natural frequency of the column is then

$$n_{\text{critical}} = \frac{30}{\pi}\sqrt{\frac{3EIg}{Wl^3}} = 325\sqrt{\frac{EI}{Wl^3}},$$

where E, I, and l are expressed in inch units and W in pounds weight.

At first glance it might seem that there is something peculiar about this expression, since it implies that the critical frequency for a column is the same as for a cantilever beam. The reason for this is that in the general formula for the period of a harmonic motion, namely

$$= 2\pi\sqrt{\frac{m}{k}},$$

m denotes the mass of the vibrating body, while k denotes the unit restoring force which in this case is the flexural rigidity of the column considered as a vertical cantilever beam with its lower end fixed.

In order to make this still clearer, the following alternative proof may be given:

When the weight W is vibrating laterally with a simple harmonic motion, the maximum value of its velocity v is at mid-position and is of amount

$$v = r\omega$$

where in the present case the amplitude $r = d$, and since the

motion is harmonic, $\omega = \sqrt{\dfrac{f}{m}}$. The kinetic energy of W in this mid position is therefore

$$K.E. = \tfrac{1}{2}mv^2 = \tfrac{1}{2}mr^2\omega^2 = \tfrac{1}{2}md^2\frac{f}{m} = \tfrac{1}{2}d^2f.$$

This energy must be equilibrated by the resilience of the column —that is, by its internal work of deformation or potential energy. If d is the amplitude of lateral deflection and F is the force required to produce this deflection, then since the column deflects laterally like a cantilever beam fixed at the lower end, we have

$$d = \frac{Fl^3}{3EI}$$

and consequently the potential energy stored in the beam at either extreme position is

$$P.E. = \tfrac{1}{2}Fd = \frac{3EId^2}{2l^3}.$$

Equating these two expressions, we find

$$P.E. = \frac{3EId^2}{2l^3} = K.E. = \tfrac{1}{2}d^2f$$

whence as before

$$f = \frac{3EI}{l^3}$$

and consequently

$$P = 2\pi\sqrt{\frac{m}{f}} = 2\pi\sqrt{\frac{Wl^3}{3EIg}}$$

The effect of including the weight of the column in the calculations may be determined approximately as follows: Since the deflection of a cantilever beam under uniform load is

$$d = \frac{wl^4}{8EI}$$

where w denotes the uniform load per unit of length, the total deflection of such a beam under both a concentrated load W and a uniform load wl will be

$$d = \frac{Wl^3}{3EI} + \frac{wl^4}{8EI} = \frac{l^3}{3EI}(W + \tfrac{3}{8}wl).$$

It is therefore the same as for a single concentrated load of amount $W + \tfrac{3}{8}wl$. Assuming that the frequency of the column is the same for a single concentrated load as when part of the

load is uniformly distributed, we have approximately in this case

$$n_{critical} = \frac{30}{\pi}\sqrt{\frac{3EIg}{l^3(W + \frac{3}{8}wl)}}.$$

A rigorous analysis of this problem shows that the fraction of the uniform load wl which should be added to the concentrated load W to obtain the true frequency, is $\frac{33}{140}$ instead of $\frac{3}{8}$.[1] Consequently the corrected formula for critical speed becomes

$$n_{critical} = \frac{30}{\pi}\sqrt{\frac{3EIg}{l^3(W + \frac{33}{140}wl)}} = 325\sqrt{\frac{EI}{l^3(W + \frac{33}{140}wl)}}$$

where W = total load in pounds supported by one column.
w = average weight of column in pounds per linear inch.
l = length of column in inches.
wl = total weight of one column in pounds.
I = static moment of inertia of cross-section of column in inches.[4]
E = Young's modulus for the material.

To determine the required size of column so as to avoid synchronism, we may assume its natural frequency $n_{critical}$ to be a certain multiple of the operating speed n assumed to be known, say

$$n_{critical} = 10n.$$

Neglecting the weight of the column as a first approximation, we have in this case

$$n_{critical} = 10n = \frac{30}{\pi}\sqrt{\frac{3EIg}{Wl^3}}$$

whence

$$I = \frac{n^2Wl^3}{1{,}057E}.$$

The dimensions of the column may then be determined so as to give approximately this value of I. Its weight may then be computed and its critical frequency recomputed using $W + \frac{33}{140}wl$ for the load.

20. Marine Installations.—The hull of a vessel is a compound beam, made up of the sides, decks and bulk heads as component girders. It therefore vibrates in the same manner as a massive beam having the same distribution of mass and moment of inertia.

In order that the motion of the engine shall not set up forced vibrations in the hull, two conditions must be fulfilled.

[1] Ibid.

A. The combined center of gravity of all the moving masses must always remain at rest relative to the hull of the vessel.

B. The algebraic sum of the moments of momentum of the moving masses must be zero at each instant for every arbitrary center of moments.

The first condition is taken care of in the design of certain types of motors. For example in a 6- or 12-cylinder gas engine, like the Liberty motor, the combined center of mass of the

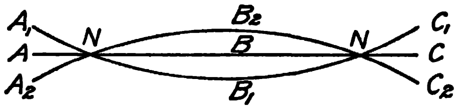


FIG. 14.

reciprocating parts is a fixed point, and consequently in this type of motor no vibration is set up by the motion of the reciprocating parts. Both of the above conditions *A* and *B* may be satisfied simultaneously for an engine with not less than four cylinders by applying the method of inertia balancing devised by Schlick.

For an ordinary marine engine with any number of cylinders these conditions may be so applied as to minimize vibration by properly locating the engine in the hull.

Consider first the effect of placing a single vertical cylinder engine at or near the center of the vessel, represented by the point *B* in Fig. 14. Let *ABC* represent a straight line in the vessel when the engine is at rest. When the engine attains a certain critical speed—namely, when its speed becomes the same as the natural frequency of vibration of the hull—strong vibrations will appear, causing the line *ABC* to vibrate between the two extreme positions $A_1B_1C_1$ and $A_2B_2C_2$. If the speed is increased beyond the critical point, the vibrations die away. The same phenomenon will occur if the engine is placed near the ends *A* or *B*—that is, near the bow or stern. However, if placed at either of the points lettered *N* in Fig. 14, which represent the nodes of the vibration for this frequency, the vibrations will not appear at this speed, but may occur at higher speeds, provided of course that it is possible for the engine to reach these speeds. In other words, the simple form of vibration shown in Fig. 14 corresponds to the fundamental, or lowest natural frequency of the hull, which is usually the only one which lies within the range of engine speeds in the case of steam engines. Any beam, however, has a whole sequence of higher natural frequencies,

called harmonics of the lowest, or fundamental, frequency, each of which possesses its own characteristic form, as well as a fixed number and location of nodes or stationary points. In high speed motors these harmonics may be the critical frequencies to be avoided.

For a two-cylinder vertical engine with cranks 180 deg. apart, the vertical inertia forces cancel, but the opposition of the cylinders give rise to an inertia couple. Consequently the best location for such an engine is at the center or ends of a vessel where the couple will have the least effect. If placed at a node, with the cylinders on opposite sides of the node and equidistant from it, the couple will have the greatest effect, producing vibrations of the first order at the lowest critical speed. As the engine is moved away from the node, the effect becomes less marked.

Any system of forces whatever can always be reduced to an equivalent system consisting of a single resultant force and a single resultant couple. This of course applies equally to the inertia forces in a machine. From what precedes, then, it is apparent that there are two general rules for avoiding vibrations of the first order in marine installations:

C. For an engine placed at the center of a vessel the resultant vertical force must be zero (i.e., the algebraic sum of the vertical inertia forces must be zero) but for this particular location the resultant inertia couple has its least effect in producing vibration.

D. For an engine placed at or near a node the resultant fore and aft couple must be zero, but it is not so important that the resultant of the vertical inertia forces shall be zero.

Consider next a three cylinder vertical engine, say a triple expansion steam engine with cranks 120 deg. apart (Fig. 15). The

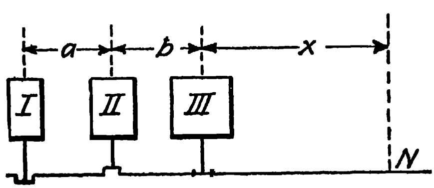


FIG. 15.

usual location of such an engine is aft of the center of the vessel, and therefore near one of the nodes of the fundamental frequency. Let W_1, W_2, W_3 denote the weights of the reciprocating parts for the three cylinders respectively and let a, b, x denote the

distances of these three cylinders respectively from the node (Fig. 15). Let φ denote the angular displacement of the crank OA from the inner dead center D (Fig. 16). Then its vertical displacement is

$$y = r - r \cos \varphi$$

Consequently its vertical velocity is

$$v = \frac{dy}{dt} = r \sin \varphi \frac{d\varphi}{dt} = r\omega \sin \varphi$$

where ω denotes the angular velocity of the shaft, assumed to be constant. For a simple approximate solution, neglect the angularity of the connecting rod. Then this expression for v will also denote the velocity of the reciprocating parts. If M denotes the mass of these parts, their linear momentum is

$$Mv = Mr\omega \sin \varphi$$

Now consider two positions of the crank as indicated in Fig. 17. Finding the momentum of each of the three sets of reciprocating weights W_1, W_2, W_3, and taking moments about the node we

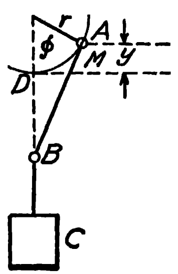


FIG. 16.

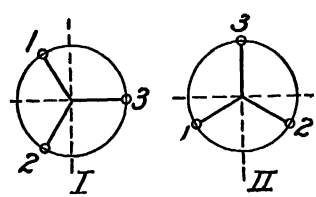


FIG. 17.

obtain from condition (D), given above, for these two positions the two following relations:

$$\frac{W_1}{g} r\omega \sin 210° (a + b + x) + \frac{W_2}{g} r\omega \sin 330° (a + x) + \frac{W_3}{g} r\omega \sin 90° (x) = 0$$

$$\frac{W_1}{g} r\omega \sin 300° (a + b + x) + \frac{W_2}{g} r\omega \sin 60° (a + x) + \frac{W_3}{g} r\omega \sin 180° (x) = 0$$

Cancelling the common factor $\frac{r\omega}{g}$ and substituting the values of

the trigonometric functions, these reduce to the following:

$$-\tfrac{1}{2}W_1(a + b + x) - \tfrac{1}{2}W_2(a + x) + W_3x = 0$$

$$-\tfrac{1}{2}\sqrt{3}\,W_1(a + b + x) + \tfrac{1}{2}\sqrt{3}\,W_2(a + x) = 0$$

Further reduction shows that these two equations are equivalent to the simple conditions

$$W_1(a + b + x) = W_2(a + x) = W_3x$$

For a triple expansion steam engine we have approximately

$$W_1 : W_2 : W_3 = .71 : .83 : 1$$

Assuming for simplicity that $a = b$, substituting the numerical values of these ratios, and solving for x we have approximately

$$x = 5a$$

If, then, the engine is installed in this location, it will produce a minimum of vibration in the hull.[1]

In practice engines are frequently installed in approximately this location, which accounts for the fact that some installations show very little vibration.

21. The Akimoff Foundation.—The usual method of supporting machinery consists in anchoring the machine down firmly to a foundation designed to be as rigid and massive as circumstances permit. As explained above, the effect of massiveness in a foundation is to reduce the amplitude of motion. However, a large mass vibrating with even a small amplitude absorbs energy which can only come from the machine, and consequently this practice results in considerable power losses. In addition to power losses, vibration is likely to cause failure of certain parts, due to fatigue of the material under repeated stress. There are many other undesirable effects, such for instance as disintegration of concrete foundations under heavy machinery such as turbo-generators, producing trouble by allowing the machine to settle out of alignment.

It is a fact of common experience that there are certain critical speeds for any machine at which vibration is greatly intensified. A partial explanation of this effect and means for avoiding it are given in what precedes. The following supplementary explanation may serve to make the matter clearer.

Every elastic body, or system of elastic members, if displaced from its position of equilibrium and then released, will oscillate about this position with a certain definite frequency depending

[1] This criterion for locating a triple expansion marine engine is due to Schlick.

on the distribution of mass and stiffness of the system. Such oscillations which a body performs of itself are called natural or free oscillations, and their frequency is called the natural frequency of the system. Free vibrations absorb very little power, as practically the only energy dissipated is a very slight amount due to molecular friction. On the other hand, any elastic body or system of members may be forced to oscillate with any given frequency by the action of an external periodic force. Such forced vibrations, however, require the expenditure of power, the amount of power required depending of course on the mass and rigidity of the system as well as on the frequency of the forced vibration.

Whenever the frequency of the forced vibration happens to be the same as the natural frequency of the system, we have what is called synchronism between the two. The result of synchronism is naturally excessive vibration, since the power required to maintain vibration with the natural frequency of the system is very slight and consequently the excess energy received from the exciting force is manifested in greater amplitude of motion, often to such an extent as to produce fracture of some part.

The critical speed then is simply that at which synchronism occurs between the frequency of the impressed force and the natural frequency of the system on which it acts, the resistance in this case dropping to a minimum, and consequently the amplitude of motion showing a corresponding increase so as to keep their product sensibly constant and proportional to the rate at which work is being done on the system.

In designing foundations for machinery it is usually very important to prevent synchronism between the operating speed of the machine and the natural frequency of adjacent structures or machines. Vibration is frequently transmitted through the walls, partitions, and floors of a building, and through the soil for long distances, making itself felt unpleasantly and perhaps dangerously whenever it extends to any structure which synchronizes with it. For example, the blades of a turbine rotor have been found in vibration although the turbine was not running, due simply to synchronism between the natural frequency of the blades and the operating speed of adjoining machinery. The function of a foundation should be therefore not only to support weight and damp out vibration, but also to prevent the occurrence of synchronism.

The Akimoff patented foundation is designed to accomplish all three of these results simultaneously. It consists primarily in interposing a three point support between the machine and the foundation proper, one of these supports being rigid and of the nature of a universal joint, and the other two supports being resilient. The three main features of this type of mounting may be briefly summarized as follows:

(*a*) *Concentrates Loads at Three Definite Points and Maintains Alignment.*—A three point support localizes the loads transmitted to the foundation at these three points. This simplifies the design of the foundation since when the exact amount and point of application of a load is known it is a simple matter to provide adequate support. Most of the difficulties in foundation design are due to uncertainty as to the distribution of the load. By making one of the three supports adjustable as to height, this provides means for leveling the machine and maintaining its alignment. A three point support also prevents the occurrence of twisting strains which often occur in certain types of machines, such for instance as automotive apparatus.

(*b*) *Permits Damping.*—The resilient supports may be so designed as to serve as dampers for absorbing the energy of vibration. The function of dampers, and their construction are explained more fully in the following article. It may be mentioned in this connection, however, that by constructing the resilient supports so as to have a large coefficient of internal friction, the energy of vibration may be largely dissipated in the damper itself and consequently disappear before it reaches the foundation. This is evidently the most effective means of disposing of vibration since the disturbance is given no chance to extend to adjacent structures.

(*c*) *Prevents Synchronism.*—The principle on which the effectiveness of this foundation depends consists not merely in using a three point support, but in making one of these points rigid and the other two elastic. The single fixed point prevents linear motion of the machine in any direction, whereas the elasticity of the resilient supports leaves it free to rotate very slightly about any axis through the fixed point. The effect of this is to allow the machine three degrees of freedom, of course within very small limits, instead of attempting the impossible task of providing an absolutely rigid anchorage. A means is thereby provided for controlling the motion of the machine through the

agency of the resilient supports so as to prevent the possibility of synchronism between the operating speed of the machine and the natural frequency of the foundation or substructure.

Fig. 18 is a sketch of the Akimoff foundation applied to a 3000 K. W. turbo-generator mounted on a structural steel framework.

22. Foundation Dampers.—In a machine anchored rigidly to a heavy fixed foundation, the mass of the foundation acts as an inertia damper to limit the displacement due to vibration, as

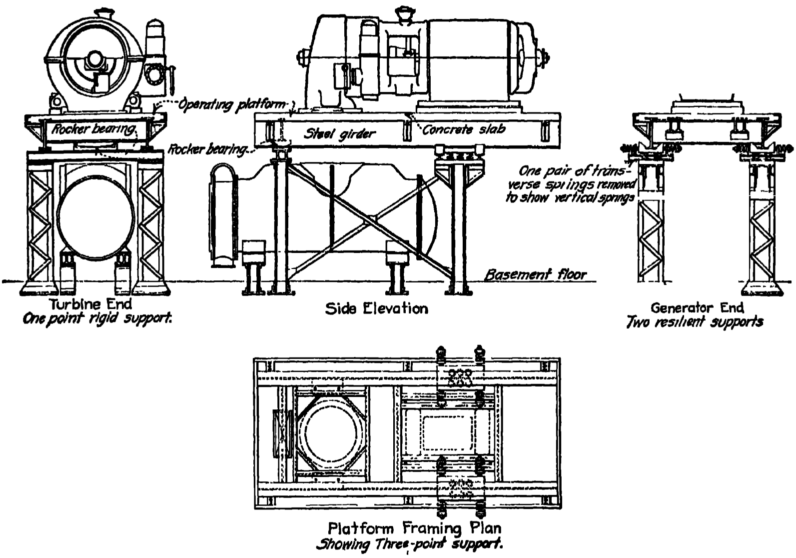


FIG. 18.

explained in Art. **16.** The smaller the ratio of the mass of the moving parts of the machine to the mass of the fixed parts of the machine, including the foundation anchored to it, so much the less will be the amplitude of vibration, since the whole question is one of transferring the kinetic energy of vibration to the foundation. From what precedes it is also apparent that not only are the size and density of the moving parts concerned, but also their position relative to one another and to the foundation on which the machine is supported. The ideal procedure in designing foundations for machinery is then to begin with the machine itself, and by proper arrangement and design of the moving parts, cancel the free inertia forces so far as possible. The location of the machine with respect to the foundation must next be considered, as for instance in the case of marine installa-

tions considered in Art. 20. The weight and distribution of mass in the foundation itself should then be so proportioned relative to the machine as to offer the most effective resistance to the kinetic reactions which are finally transmitted to the foundation.

In most cases, however, this procedure cannot be followed, as the type of machine and foundation are both fixed by other considerations. For instance, in automotive apparatus, the motor and its support are designed for compactness, lightness, and appearance; or, when installing standard machinery on the upper floors of old buildings, where it is impossible to build up a massive foundation, it is necessary to resort to other means for disposing of the free kinetic energy of vibration, say by transferring it to an elastic system where it can be absorbed or damped out by internal frictional resistance.

A method frequently employed for this purpose consists in placing the machine on a layer of some resilient material such as cork or felt, called an "insulating" layer. This of course is a very simple method to apply, but it is only partially effective. In fact this should not be called "insulation" since this term by reason of its common use in electrical work has come to mean a layer which is impenetrable to a certain type of wave motion. The first fact to observe is that a resilient layer is only effective in proportion to its capacity to absorb work by internal or molecular friction. As this capacity for any given layer is limited, it is capable of absorbing only a certain fraction of the energy of vibration, and it is usually not practicable to introduce sufficient volume to absorb all of it. Furthermore, this damping layer may not be inserted at any arbitrary place, for as shown above, the amplitude of vibration depends on the ratio of the mass of the moving parts to that of the foundation and fixed parts. Since inserting a resilient layer diminishes the effective mass of the foundation, these layers should therefore be placed at such a depth that the machine will still be attached to a sufficiently heavy foundation mass. Moreover, inserting a resilient layer has the effect of raising the center of gravity of the machine and therefore affects its stability, which must also be taken into account in determining the position of such a layer. This leads to the principle that when practicable, instead of isolating each machine separately, several machines should be built on a common support, and this support isolated from its surroundings by resilient layers, or some form of damper as described below.

In order to be effective as a vibration damper, a resilient layer should be lightly loaded, the allowable weight per square inch depending of course on the nature of the material used for this purpose, as the damping action depends entirely on preserving the elasticity of the material and preventing permanent deformation. The layer should also be impregnated with some preservative so as to make it impervious to water, oil, acids, etc., in order that it may permanently retain its damping properties. The use of a simple resilient layer is shown in Fig. 19.

Naturally, absorption of vibration is not limited to entire machines. Absorption dampers may also be applied to separate

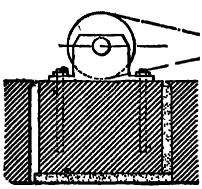


Fig. 19.

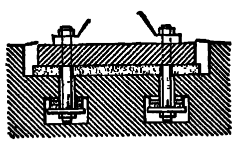


Fig. 20.

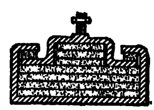


Fig. 21.

parts such as machine feet, brackets, and other kinds of supports. As such dampers are intended for special purposes, they must be constructed in a special manner. Figs. 20 and 21 illustrate two simple types of dampers for machine feet. These are so designed as not to change the height of the machine, and also to act as an anchorage against vertical upward pull as well as downward compression. The use of a metal housing as shown in Fig. 21 is important, as it prevents the resilient material from lateral deformation and thereby increases its efficiency.

It is often advisable to use springs in connection with absorbent material. One form of a combination damper of this type is shown in Fig. 22. The springs in such dampers absorb very little energy as their molecular friction is very slight provided the load does not exceed the elastic limit of the steel. The spring, however, does decrease the intensity of vibration since it extends the action of the force over a greater period of time. For this reason a combination damper should be used in a machine subject to shocks. For instance in an automobile, in addition to the vibration caused by the motor there are also severe road

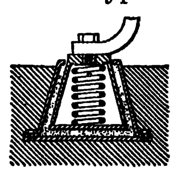


Fig. 22.

shocks to be considered. Therefore a combination damper is desirable in this case since the action of the spring serves to relieve shocks as well as to reduce the intensity of vibration, while the resilient material dissipates the energy of vibration in proportion to its volume and specific damping properties. Whenever springs are used, however, care must be taken that the speed of the machine does not synchronize with the natural frequency of the spring, as in this case the effect will be exactly the reverse of that desired.

SECTION 7

BRIDGE PIERS AND ABUTMENTS

GENERAL CONSIDERATIONS

BY PHILIP GEORGE LANG, JR.

1. Selection of Site.—The selection of the site of a bridge is necessarily regulated, to a large extent, by the topographic and geologic features of the vicinity, as these will influence the selection of an economic crossing and the general type of construction. The proposed location should be subjected to a careful and intelligent examination, all local conditions ascertained and recorded.

2. Survey of Local Conditions.—The profile of the ground, and also the bottom of the stream, should be carefully plotted, the rate of stream flow determined, the periods and elevations of high and low water as indicated by the records of the past ascertained, and the area of the waterway determined for which it is necessary that provision be made. The channel proper should be located, and, in the case of bridges consisting of two or more spans, the location of the channel span should be so fixed as to provide minimum obstruction.

3. Foundations, Borings, Etc.—Especial attention should be given to the determination of the character of the natural foundations on which the substructure will rest. When small bridges are under consideration, the digging of test pits usually supplies adequate and reliable data. Where large and costly structures are contemplated, test borings should be made. This work requires equipment of a class whose possession is usually restricted to firms actually engaged in this class of work, and the most economic and satisfactory results can usually be obtained by contracting for borings of this nature. Core borings to any desired depth can usually be contracted for at prices ranging from $1 to $3 per ft., depending upon the character of the soil, the depth to which it is necessary to bore, and whether the site of the boring is on land or water.

4. Outside Interests.—*Effect on Design.*—Where the proposed bridge involves the crossing of a navigable stream, it is necessary that the navigation interests be consulted, their wishes and needs ascertained. Where the proposed structure is to cross a highway or tracks, the desires of the interests whose right-of-way is crossed must receive due consideration, and suitable agreement covering the rights of each party to the crossing should be formulated. In such cases, the general economic considerations which fix the location of the substructure units and the superstructure span lengths are frequently modified by the demands of the interests controlling the property crossed. Frequently the desire to avoid or minimize the obstruction upon such property requires an arrangement of piers and abutments which is not the most economical from the standpoint of bridge costs. The angle of the crossing is also an element which cannot be neglected. It is probably needless to say that the most economical arrangement is a right-angle crossing. Where, for any controlling reason, this is impossible, the intersection should be made as near a right angle as possible.

In the case of bridges crossing interstate navigable streams, the approval of the United States War Department must be obtained prior to the beginning of actual construction. Governmental regulations covering such instances require that the application for War Department permit be made on standard form, accompanied by the plans and supporting documents enumerated in the instructions therein printed. The most satisfactory method of conducting negotiations for the issuance of a War Department permit is usually by personal conference with the Engineer Officer of the War Department in whose district the proposed work lies.

In many parts of the United States, public regulatory bodies of various kinds exist, exercising statewide or local jurisdiction over various phases of bridge construction—the capacity of the proposed new structure, its adaptability to the local conditions and the traffic which it is to carry, waterway provided, æsthetic features, etc. In some cases, the formal approval of such bodies is required by law. Under any circumstances, it is a matter of sound policy that these bodies be consulted.

5. Economic Balance of Cost.—When all of the factors influencing the character of structure to be adopted have been fully considered, it may be worthy of note, as a statement frequently

repeated, that the most economical type of bridge is that in which the cost of the piers and abutments—that is, the substructure—approximately equals the cost of the superstructure. This general statement, however, is subject to wide variations. The ratio between the cost of the substructure and that of the superstructure varies considerably. Extremes may be represented by the profile of the crossing—that is, its characteristics may vary between a broad, low crossing or a crossing which is narrow and high. A further factor to be considered is the anticipated life of the superstructure and that of the substructure. In general, it may be assumed that the substructure will have an anticipated life at least twice that of the superstructure.

6. Concrete Construction.—The discussions in this section treat only of concrete construction, whose economy over stone masonry has been clearly established by engineering experience. This fact is attributable to the large labor cost involved in the latter class of construction, the current scarcity of expert stone cutters, and the impossibility, in many localities, of securing a satisfactory quality of stone except at prohibitive cost, in which transportation is a factor. Concrete as a material of mass construction has been in use for such a length of time that its nature and composition are matters of common and thorough knowledge. The plant and equipment required for such construction does not necessitate the employment of highly skilled labor, and all material may be readily transported and handled. In many instances, sand and stone are secured locally, and, after mixing, the concrete may, with proper plant arrangement, be placed in final position by chutes, or other means, at comparatively small labor cost.

Only mass or monolithic concrete, in contradistinction to reinforced concrete construction, will be discussed in detail. The external forces acting on piers and abutments for bridge substructures are identical, whether the structure is of the monolithic or reinforced type. A reinforced pier or abutment may be considered a framed structure, consisting of columns, beams, slabs, counterforts, etc., all of which must be designed in accordance with the principles laid down for framed reinforced concrete construction, and, except as previously mentioned for external forces, such piers and abutments differ radically from the type under consideration. Furthermore it is desired to call attention to the fact, that, in the case of reinforced concrete construction,

it is vitally essential that great care be taken in the design to provide proper drainage, and, in addition, thoroughly waterproof the structure, in order to prevent deterioration of the reinforcing material. Integral waterproofing will not do this satisfactorily, and it becomes necessary to resort to membrane waterproofing, which must, in turn, be protected.

7. Bridge Seats.—The tops—that is, the bridge seats—of piers and abutments provide the immediate bearing surface for the support of the superstructure. The superstructure, especially in the case of railroad bridges, and, to a somewhat minor degree, in the case of highway bridges subject to the movement of heavy trucks or trolley traffic, is subject to vibration, which must be absorbed by the supporting masonry. It is essential that the bridge seats be composed of a good quality of concrete, certainly not leaner than 1 : 2 : 4 mixture.

8. Protection of Pier Surfaces.—In the case of bridges crossing waterways, it is necessary to consider especially that portion of the substructure located "between wind and water"—that is, the portion of the masonry surface which lies between extreme high and extreme low water. This surface is subject to injury and consequent deterioration, due to impact of objects floating on the water, the erosive action of the current, waves, frost, and possibly, in the case of salt water, the action of the saline content. In regions where industrial plants or coal mines abound, this danger to the masonry is further increased by the introduction of chemical agents, especially sulphuric acid, into the water, very small quantities of which will cause comparatively rapid deterioration of the concrete.

It is recommended that, where a leaner mixture is used for the bodies of piers or abutments, the surface for at least an approximate thickness of 2 ft., between a line 2 ft. above ordinary high water and 2 ft. below ordinary low water, be composed of a concrete not leaner than 1 : 2 : 4 mixture, thoroughly spaded, and the forms left in place at least 30 days before the concrete is exposed.

In cases where conditions are unusually adverse, the surface of the substructure between a line 2 ft. above ordinary high water and 2 ft. below ordinary low water should be formed of pre-cast concrete blocks, 1 : 2 : 4 mixture, which have set at least 30 days before being placed. Mechanical bonds should be provided to properly bond the blocks together, and to the mono-

lithic concrete forming the bulk of the masonry. The joints between the blocks should be thoroughly and completely grouted. If it is possible to obtain a satisfactory grade of stone at reasonable cost, stone may be substituted for the concrete blocks.

ORDINARY BRIDGE PIERS

BY PHILIP GEORGE LANG, JR.

9. External Forces.—The external forces to which a pier will be subjected, and to resist which it will be designed, must be determined in advance. Some of these forces are susceptible of absolute mathematical determination—the source, effect, and disposition of others is a matter of engineering judgment and experience. In pier construction these forces are:

(1) The dead load of the superstructure.
(2) The live load of traffic passing over the bridge.
(3) The dead load of the pier itself.
(4) Lateral forces, acting in a direction parallel to the center line of the pier—that is, those which act in a transverse direction with respect to the longitudinal axis of the bridge. Among these forces may be enumerated the wind on traffic passing over the bridge, the wind on the superstructure, centrifugal force in the case of a railroad bridge on a curve, force of the water current, the force due to large fields of floating ice, and the effect of impact of objects floating upon the water surface.
(5) Longitudinal forces, acting in a direction transversely to the center line of the pier—that is, those which act in a direction parallel to the longitudinal axis of the bridge. In the case of a railroad bridge, it is necessary to consider such forces which are caused by the stopping and starting of trains.

The impact produced by the live load of traffic passing over the bridge can usually, in the case of substructures, be neglected. In any event, it will be a small portion of the total load, even though the impact effect is not entirely dissipated in the superstructure and substructure before it reaches the foundations.

Buoyancy must be considered in the design, particularly if there is any possibility of a combination of forces whereby the stability of the pier, considering buoyancy, may be compromised. It rarely happens that such is the case, and generally the neglect of buoyancy will be on the side of safety, as such neglect will increase the calculated loading on the foundation.

Wind pressure is to be taken at 30 lb. per sq. ft. on 1½ times the exposed area of the superstructure as seen in elevation, and also on the exposed area of the traffic passing over the structure.

Centrifugal force, in the case of railroad bridges, should be provided for in accordance with the American Railway Engineering Association Specifications, as follows: on curves, the centrifugal force (assumed to act 6 ft. above the rail) shall be taken equal to a percentage of the live load, including impact, according to the following table:

Degree of curve	20′	40′	1°	2°	3°	4°	5°	6°	7°	8°	9°	10°	11°	12°
Percentage	2.5	5	7.5	10	10	10	10	10	10	10	10	10	10	10
Speed (miles per hour)	80	80	80	65	53	46	41	38	35	33	31	29	28	27

In the case of highway bridges, it is unnecessary to make provision for centrifugal force.

The force due to the water current is to be assumed equal in pounds per square foot to $P = 1.5v^2$, "V" being the velocity of the current in feet per second. This is to be used for flat surfaces; for rounded surfaces, use one-half of the above quantity.

The pressure due to a floating field of ice may, as an extreme, be taken equal to the crushing strength of ice, which varies from 300 to 800 lb. per sq. in., 43,200 to 115,200 lb. per sq. ft. It is unnecessary to consider ice as more than 1 ft. thick.

The longitudinal force on railroad bridges shall be considered as equal to 20 per cent of the live load on one track only, applied 6 ft. above the top of rail.

All of the foregoing forces must, in each case, be carefully determined, the effect of each calculated, and the resultant computed, so that the most severe effect on the foundations is obtained. The pier, in any event, must be designed so that there is no uplift, as the masonry cannot be anchored in a satisfactory manner to the natural foundation encountered. The effect of this is that, in the case of rectangular areas, the resultant of all possible forces must fall within the middle third. These forces are ultimately resisted by the natural foundations encountered. Spread or stepped footings must be provided where necessary for their distribution to the underlying material, and where pile foundations are used. The stepping should preferably be at an angle of 60 deg. with the horizontal—never less than 45 deg.

10. Location.—The pier should be so located with respect to the superstructure that the center of gravity of the superimposed load—that is, the live and dead load of the superstructure—will be coincident with the center line of the pier.

11. Top Dimensions.—In pier design, the top dimensions are first to be fixed. These dimensions are determined by the character, width, and length of the bridge. The width of the pier is dependent upon the size of the bearing plates or shoes upon which the superstructure rests, and should not be less than 2 ft. more than the out to out dimension of the bearing plates or shoes, measured along the longitudinal axis of the superstructure.

The length of the pier should, in no case, be less than 4 ft. in excess of the extreme width of the supported superstructure, measured from out to out of bearing plates. The bearing of the superstructure on the substructure should be so that the dead and live load of the superstructure does not exceed 600 lb. per sq. in.

12. Foundation Dimensions.—The dimensions of the foundations are primarily dependent upon the character of the underlying material—that is, the load in tons per sq. ft. which it has been determined in advance this material will be capable of not only safely supporting, but supporting without undue settlement.

13. Batter.—The surfaces of the piers should be battered ½ in. per ft. This may, in extraordinary cases, be increased as necessary, in order to secure proper stability.

14. Pier Ends.—Piers in streams constitute an obstruction to the waterway, and increase the liability to scour. This tendency can be materially reduced by due attention to the form of pier end, which should, in general, be so shaped as to afford the minimum obstruction to the stream flow. Experience and tests indicate that, for both the upstream or nose end and the downstream or tail end of piers, the half-round—that is, semi-circular shape—affords the minimum obstruction to the waterway consistent with practical construction. As a second choice, a 45-deg. nose and tail are recommended.

15. Nose Protection.—In localities where heavy ice movement occurs, where exceptionally rapid stream flow exists, or where a combination of both conditions may be anticipated, it is advisable to build a 45-deg. nose, this nose to be protected from a point

not less than 4 ft. above extreme high water to 4 ft. below extreme low water by an angle iron, with properly designed bent bolts inserted at intervals not greater than 2 ft., these bolts being embedded in the concrete, and the angle iron nose protection thus held in place.

16. General.—The body of the pier should be composed of a mixture not leaner than 1:3:5 concrete. One man stone may be used in the foundations, if the stones are completely embedded and entirely surrounded by concrete. It will be found advantageous to use a small amount of reinforcing near the surface of the pier as a framework for the attachment of wire mesh. This will have the effect of providing a mechanical bond in the concrete, and reduce the probability of surface cracks, due to temperature or other causes.

The disposition and treatment of piers which support bridges crossing public highways, steam, or electric railroad tracks is necessarily governed by the local conditions and the wishes of the local authorities or other interests having jurisdiction. It is advisable that, where piers are placed between tracks, they be protected by a nose or buffer to prevent damage to the structure in the event of derailment.

17. Pier Dimensions.—For piers supporting railroad bridges, the following sizes represent good practice:

Square End Piers—(Dimensions taken at undercoping)
- Single-track deck plate girders—6 × 16 ft. and 8 × 16 ft.
- Double-track deck plate girders—6 × 29 ft. and 8 × 29 ft.
- Single-track through plate girders—6 × 24 ft. and 8 × 24 ft.
- Double-track through plate girders—6 × 37 ft. and 8 × 37 ft.

Round End Piers—(Width at undercoping—length center to center of ends at undercoping)
- Single-track deck plate girders—6 × 14 ft. and 8 × 14 ft.
- Double-track deck plate girders—6 × 27 ft. and 8 × 27 ft.
- Single-track through plate girders—6 × 22 ft. and 8 × 22 ft.
- Double-track through plate girders—6 × 35 ft. and 8 × 35 ft.

18. Quantities.—The following tables give the quantities for square-end and round-end piers, as shown by Figs. 1 and 2, in various widths, lengths and heights, and include quantities for 5 ft. foundation, as indicated.

SQUARE-END PIERS—CUBIC YARDS OF MASONRY
(See Fig. 1)

Height (H)	Width (W) × length (L)							
	6 × 16 ft.	6 × 29 ft.	6 × 24 ft.	6 × 37 ft.	8 × 16 ft	8 × 29 ft.	8 × 24 ft.	8 × 37 ft.
8	60	105	88	133	77	135	113	170
14	88	153	128	194	112	197	164	249
20	118	206	172	260	151	263	220	332
26	151	263	220	331	192	333	279	421
32	187	323	271	409	236	408	342	513
38	226	389	327	490	283	488	410	613
44	268	459	385	577	334	572	481	718
50	313	534	450	670	388	662	558	821

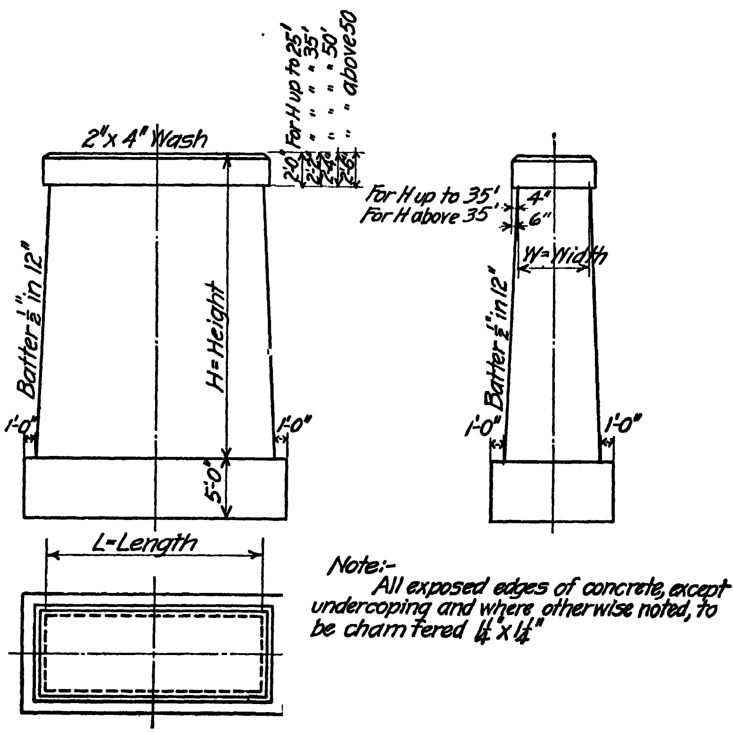


FIG. 1.

Round-end Piers—Cubic Yards of Masonry
(See Fig. 2)

Height (H)	Width (W) × length (L)							
	6 × 14 ft.	6 × 27 ft.	6 × 22 ft.	6 × 35 ft.	8 × 14 ft.	8 × 27 ft.	8 × 22 ft.	8 × 35 ft.
8	72	117	100	145	99	157	136	194
14	104	170	145	210	143	227	196	280
20	138	227	193	280	190	302	260	372
26	176	288	245	357	240	382	328	470
32	217	353	300	437	293	456	400	573
38	260	423	360	524	350	555	478	681
44	307	496	424	615	411	650	560	798
50	356	577	492	712	474	718	644	917

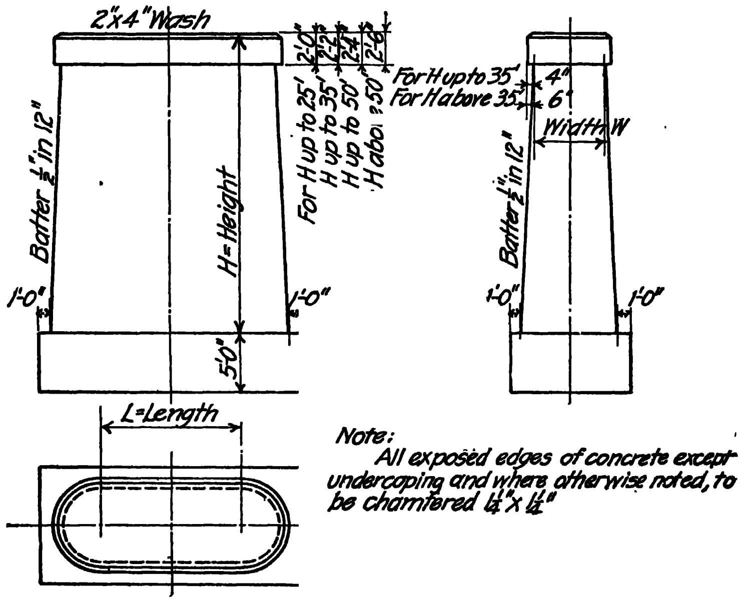


Fig. 2.

ABUTMENTS

By Philip George Lang, Jr.

19. Structural Elements.—Every abutment is, in general, composed of three distinct structural elements, namely:

(1) The breast, which directly supports the dead and live load of the superstructure, and serves for the retention of the material deposited in its rear.

(2) The wings, which are, in reality, extensions of the breast, and furnish no support for the superstructure, but act as retaining walls to prevent the encroachment of the material deposited behind the abutment upon the area or passageway in front.
(3) The backwall, which is a small retaining wall, preventing the material in back of the abutment from flowing on to the bridge seat, that part of the breast which supports the superstructure.

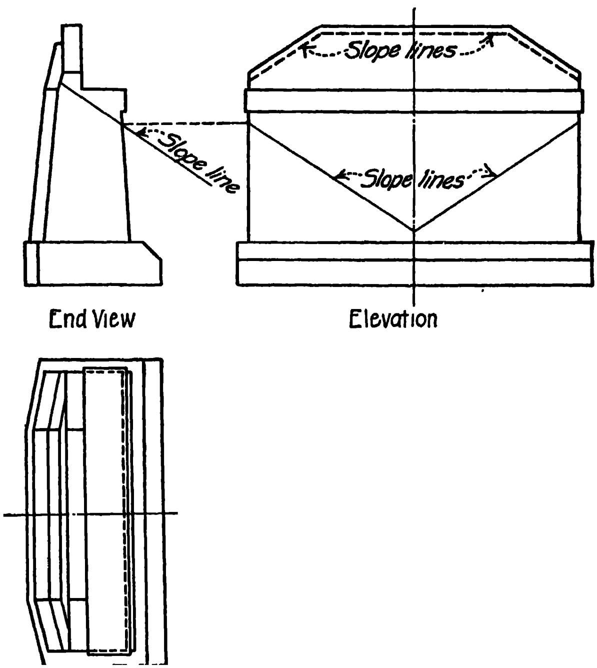


Fig. 3.—Breast abutment.

20. Breast Abutment.—The simplest form of abutment is that without wings, composed solely of the breast and backwall. Such an abutment may be likened to a pier, whose thickness has been increased to resist the lateral thrust of the material in back of it, and which has been provided with a suitable toe in front, for the purpose of increasing its stability. Some of the retained material flows around in front of the abutment (see Fig. 3).

21. "T" Abutment.—The "T" abutment is essentially a pier, reinforced in the rear by a stem which supports the superimposed load, and whose length is made such that the toe of the slope of

the retained material is at or behind the front face of the pier (see Fig. 4).

22. "U" Abutment.—This form may be considered a "T" abutment, whose stem has been divided along the center line of the bridge, and the two halves shifted to the ends of the pier; or it may be considered a wing abutment, whose wings have been

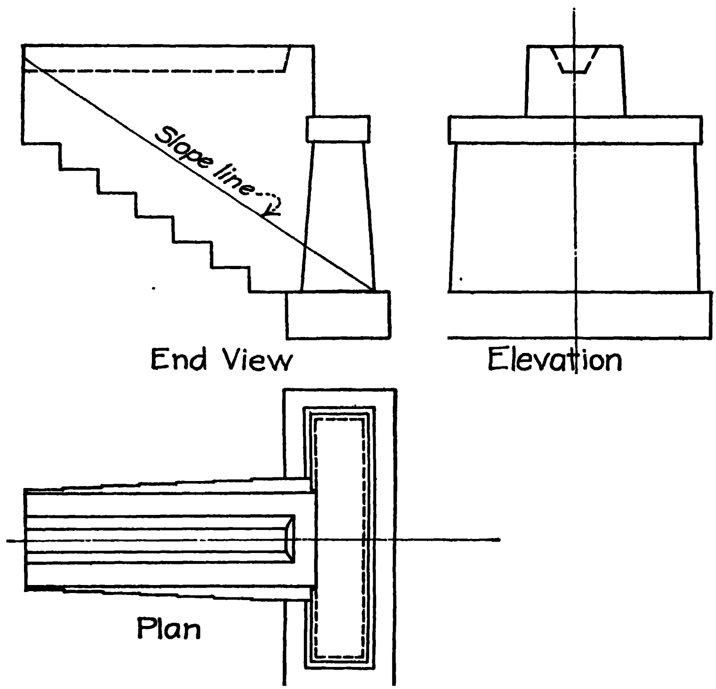


FIG. 4.—"T" abutment.

folded back parallel to each other and parallel to the longitudinal axis of the bridge. The walls in this case are made of sufficient length so that the toe of the slope is at or in the rear of the front face of the abutment (see Fig. 5).

23. "Pulpit" Abutment.—This is a modified form of the "U" abutment, which is sometimes adopted in the case of high abutments. In this form, the wings are made only of sufficient length to prevent the retained material from flowing on the bridge seat, but not of sufficient length to prevent it from flowing in front of the abutment (see Fig. 6).

24. "Wing" Abutment.—In this type, the tops of the wings are sloped to conform to the natural slope of the retained material, and the angle of the wings with respect to the normal to the longitudinal axis of the bridge made to conform to the local conditions. The most economic form is obtained when the

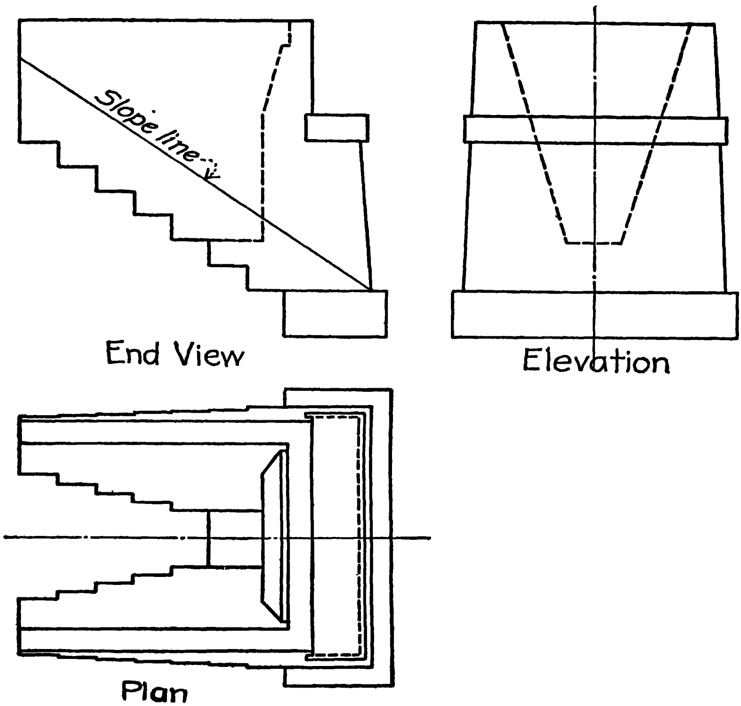


FIG. 5.—"U" abutment.

wings are made normal to the longitudinal axis of the bridge. The most common form, perhaps, is that in which the angle of wings to the normal is 30 deg. (see Fig. 7).

25. General.—Of these forms of abutments, the wing abutment is the most generally used. The pulpit abutment is used to a limited extent, principally at the ends of steel viaducts, where fill is made for the approaches, and where there is no objection to the material flowing in front of the abutment. The "U" abutment finds a limited use. Its use under certain local conditions is very satisfactory. This is especially the case where good rock foundation is found near the surface of the

ground, and particularly where the rock slopes, in which case the foundation for the wings of a "U" abutment can be readily stepped to conform to the natural slope line, thus making a material saving in the quantity of masonry. The "T" abutment was widely used during the early period of railroad construction. Since 1880, however, it has gradually ceased to be used, and, at

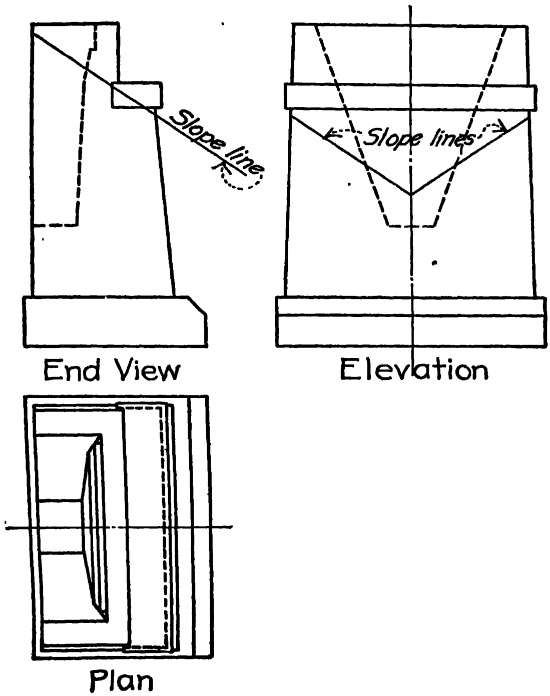


FIG. 6.—Pulpit abutment.

the present time, is rarely, if ever, constructed. The solid stem of the "T" abutment introduces undesirable conditions, particularly so far as the riding of the track in the case of a railroad is concerned.

26. External Forces.—The external forces to which an abutment will be subjected, and to resist which it will be designed, must be determined in advance. Some of these forces are susceptible of absolute mathematical determination. The source, effect, and disposition of others is a matter of engineering judgment and experience.

In abutment construction, these forces are:

(1) The dead load of the superstructure.
(2) The live load of traffic passing over the bridge.
(3) The dead load of the abutment itself.
(4) Thrust, that is, the thrust of the material retained by the abutment, in conjunction with which must be considered a surcharge allowance equivalent to the effect of the live load on the fill at the rear of the abutment.

27. Retaining Wall Characteristic.—The most troublesome factor in connection with the design of an abutment is the determination of the magnitude, direction, and point of application of

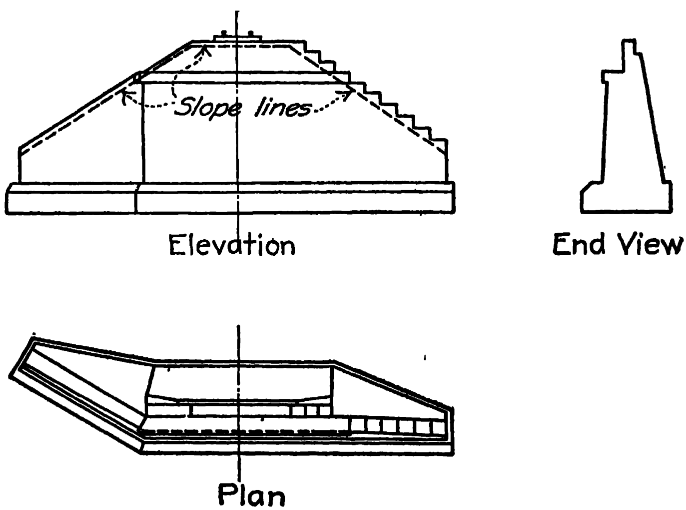


FIG. 7.—Wing abutment.

the thrust of the retained material against the abutment. The varying character of the material deposited behind and retained by an abutment introduces an element of uncertainty, and, in addition, the change in character of material, due to varying amounts of moisture in it must be considered—that is, it is conceivable that, under certain conditions of moisture, the material retained will act as a semi-fluid. On the other hand, good stiff clay, under certain conditions, may act as a solid. Attempt to reduce to a matter of mathematics the thrust exerted originated with Coulomb, who published the theory which bears his name in 1784. This was followed by Weyrauch, Rankine, and others. In general, all theories involving mathematical

investigation of earth pressures rest upon three postulates, namely:

(1) That the surface of rupture is a plane.
(2) That the application of the lateral thrust occurs at a point one-third of the height of the wall from its bottom.
(3) That this lateral force is exerted in a certain direction, actually unknown, but as a factor in mathematical process variously assumed as an essential hypothesis to the demonstration of the theory.

The entire theory of earth pressures is based, to a certain extent, upon the consideration or contemplation of the action of a liquid. The liquefication of the retained material not only increases its weight, but very materially increases the thrust, and, in the case of a liquid, the thrust is horizontal. Due to these considerations, material in back of abutments should be carefully placed and properly drained. Drainage can be readily accomplished by placing rock fill immediately back of the abutment and proper drain pipes at the bottom.

In the construction of retaining walls, and in the solution of this problem as a part of abutment design, the results of past experience must be carefully considered. An abutment whose thickness at any horizontal section is made 0.45 of the height from that section to the top of the abutment, will give a safe structure.

It should be borne in mind, in this connection, that substructures are to be considered as permanent works, and that it is false economy to use a minimum of material, based upon a purely theoretical design. In the interest of real economy, and considering the long anticipated life of such a structure, the designer should plan an abutment having a thickness perhaps in excess of the theoretical requirements, since but a small additional first cost will be incurred.

Further, he must visualize all of the conditions surrounding the reconstruction of an abutment which has, for any cause, failed in service, realizing that, in that event, it will be necessary to provide temporary supports for the existing superstructure, remove the useless masonry and properly reconstruct it. In view of these circumstances, it is believed that a thickness equal to 0.45 of the height is not excessive.

28. Dimensions.—The length of an abutment—that is, the length of the breast—is determined by the width of the approach roadway to be retained and the width of the bridge superstructure

which is to be supported. In no case should the length of the bridge seat be less than the width of the superstructure, measured over the sole plates or shoes, plus 4 ft. The width of the bridge seat is the distance from the face of the black-wall to the under-coping line, and should be not less than 1 ft. 6 in. more than the length of the shoes or sole plates. The front face of the abutment should have a batter of not less than ½ in. per ft., and preferably 1 in. per ft. A plumb face is not desirable, and should be avoided wherever possible, although in the case of an abutment whose face is coincident with the building line of a street, or under other exceptional conditions, its adoption may be necessary.

29. Wings.—The width of the wing walls at the top should be made not less than 2 ft., preferably 2 ft. 6 in. The thickness at the neat line should be made 0.45 of the height from that line to the top. From a practical standpoint, it is preferable that the tops of the wing walls be stepped to conform to the slope, although, in the case of concrete walls under discussion, the objection is often advanced that this involves an apparent attempt to imitate stone masonry, whereas concrete readily leads itself to the formation of a sloped surface. Cases may occur, especially where the face of the abutment coincides with the building line of a street, or other thoroughfare, in which it is desirable that the surface of the wall be sloped. It is to be noted, in this connection, however, that such a surface is a constant invitation to children to use it as a sliding board. This condition, has, in some instances, resulted in serious accidents. On the other hand, a stepped wall is of practical utility in the event that it becomes necessary to renew the superstructure, in which case a stepped wall affords a very suitable foundation for any falsework which may be necessary.

30. Back-wall.—The back-wall should be straight, in order that longitudinal access to the bridge seat may be unobstructed. This arrangement is of special utility in cases where renewal of the superstructure is necessary, in which event the old span may be removed and the new one placed without interference with the masonry, or undue interruption to traffic. The width of the back-wall at the top should not be less than 2 ft. and should preferably be made 2 ft. 6 in. The thickness of the back-wall at the bridge seat elevation should be made 0.50 of the height from the bridge seat to the top. In no case should this be less than the thickness at the top plus 8 in. In railroad bridges, and also

highway bridges, it will be found convenient to place a step on the back at the top of the back-wall. This step should be not less than 8 in. wide and 2 ft. 6 in. below the top of the back-wall. This arrangement will provide a footing for any temporary structures which it may be desired to place in order to support traffic before the fill is placed, or if, in future, it should become necessary to remove the fill (see Fig. 8).

31. Toe.—A properly designed toe should be used in front of the abutment breast and in front of the wing walls. It is,

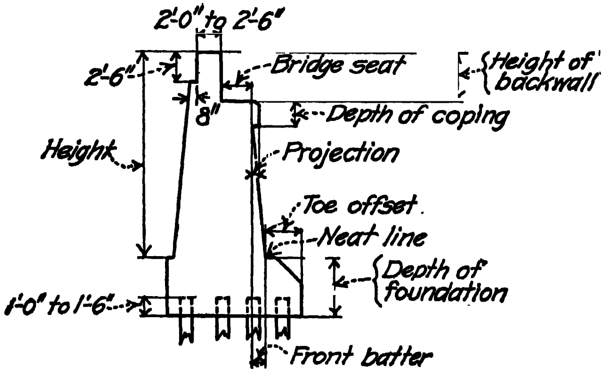


Fig. 8.

perhaps, needless to state that, in any event, the resultant of all forces at the foundation must fall within the middle third, otherwise uplift will be produced, and, since masonry cannot be anchored to the natural foundation encountered, masonry will be wasted. A good toe will, to a large extent, remedy this condition.

32. Dimensions.—The following dimensions for depth of foundation and offset for toe have been extensively used, and found to give satisfactory results (see Fig. 8):

Distance from top of abutment to neat line	Horizontal offset for toe	Depth of foundations
1′ to 20′	2′6″	4′6″
20.1′ to 25′	3′0″	4′6″
25.1′ to 30′	4′0″	5′6″
30.1′ to 40′	4′6″	6′0″

Where rock foundation is available, the toe may be omitted. For abutments less than 20 ft. in height, the toe may be modified, if conditions justify. Abutments over 40 ft. in height should be the subject of special investigation. Where rock is less than 4 ft. from the neat line, a 12 in. offset should be used, and a 12 in. offset introduced for each additional 4 ft. of foundation. The tops of all piles should project at least 1 ft. and preferably 1 ft. 6 in. into the bottom foundations.

33. Coping.—The following practice is recommended with respect to the depth of coping on concrete abutments and piers:

Height of abutment or pier	Depth of coping
Up to 25 ft....	24 in.
25 to 35 ft.....	26 in.
35 to 50 ft.....	28 in.
50 ft. and over.	30 in.

For heights up to 35 ft. use 4-in. projection; for heights in excess of 35 ft., use 6-in. projection.

34. Quantities.—The following tables give the volumes of wing abutments in cubic yards, as shown by Figs. 9 and 10, and should be used in conjunction with the accompanying sketches.

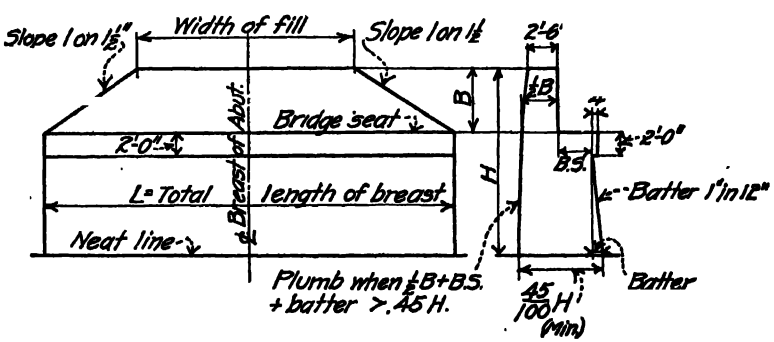


FIG. 9.

ABUTMENT QUANTITIES

18′ 6″ Width of Fill. Volumes in Cubic Yards. One Abutment with Back-wall Exclusive of Foundation and Wings

Height = H, ft.	B = 4 ft. 0 in.		B = 5 ft. 0 in.		B = 6 ft. 0 in.		B = 7 ft. 0 in.		B = 8 ft. 0 in.		B = 9 ft. 0 in.		B = 10 ft. 0 in.	B = 11 ft. 0 in.	B = 12 ft. 0 in.	Height = H, ft.
	L = 28.5 ft. Volume in back-wall = 8 cu. yd.		L = 31.5 ft. Volume in back-wall = 11 cu. yd.		L = 34.5 ft. Volume in back-wall = 15 cu. yd.		L = 37.5 ft. Volume in back-wall = 20 cu. yd.		L = 40.5 ft. Volume in back-wall = 26 cu. yd.		L = 43.5 ft. Volume in back-wall = 34 cu. yd.		L = 46.5 ft. Volume in back-wall = 43 cu. yd.	L = 49.5 ft. Volume in back-wall = 53 cu. yd.	L = 52.5 ft. Volume in back-wall = 64 cu. yd.	
	3 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	3 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	3 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	3 ft. 0 in. Br St.	4 ft. 0 in. Br. St.	3 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	3 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	
10	48	54	48	54	...	...	...	...	...	...	.....	.....	.....	.....	.....	10
12	61	70	62	71	63	68	67	74	...	...	.....	.....	.....	.....	.....	12
14	75	86	78	88	79	89	86	96	91	100	96	104	.....	.....	.....	14
16	91	102	96	106	99	109	106	123	114	126	121	133	138	142	.....	16
18	112	120	118	126	123	130	132	139	140	151	147	162	170	178	183	18
20	135	143	142	151	149	158	160	169	170	179	180	191	203	215	224	20
22	159	168	168	178	177	185	190	201	203	214	216	226	236	252	265	22
24	185	195	197	203	207	219	224	235	239	251	254	266	279	293	307	24
26	213	225	227	239	240	253	259	272	278	291	296	310	325	342	356	26
28	243	255	260	273	275	289	297	312	319	334	341	356	375	395	412	28
30	275	288	294	309	312	328	338	354	363	380	388	405	427	450	472	30
32	310	323	331	347	352	369	381	399	410	428	438	456	483	510	535	32
34	344	360	370	387	394	412	427	446	459	479	491	511	541	572	601	34
36	381	398	411	429	438	457	475	495	511	532	547	569	603	638	671	36
38	421	439	454	473	485	505	526	547	566	589	606	629	668	707	744	38
40	462	481	499	519	534	555	579	602	624	648	668	693	736	779	821	40
42	505	525	546	568	585	608	635	659	684	709	733	768	807	855	901	42
44	550	572	596	618	638	663	693	718	747	774	800	828	881	934	985	44
46	597	620	647	671	694	720	753	781	812	841	871	901	958	1,016	1,073	46
48	646	670	701	726	752	779	817	845	881	911	944	976	1,038	1,102	1,163	48
50	697	721	756	782	812	841	882	912	952	983	1,021	1,054	1,122	1,191	1,258	50

For skewed abutments, multiply respective quantities by secant of skew angle. Abutments above heavy line have a base greater than 0.45H.

ABUTMENT QUANTITIES

31′ 6″ Width of Fill. Volumes in Cubic Yards. One Abutment with Back-wall Exclusive of Foundation and Wings

Height = H, ft.	B = 4 ft. 0 in.		B = 5 ft. 0 in.		B = 6 ft. 0 in.		B = 7 ft. 0 in.		B = 8 ft. 0 in.		B = 9 ft. 0 in.		B = 10 ft. 0 in.	B = 11 ft. 0 in.	B = 12 ft. 0 in.	Height = H, ft.
	L = 41.5 ft. Volume in back-wall = 13 cu. yd.		L = 44.5 ft. Volume in back-wall = 17 cu. yd.		L = 47.5 ft. Volume in back-wall = 22 cu. yd.		L = 50.5 ft. Volume in back-wall = 29 cu. yd.		L = 53.5 ft. Volume in back-wall = 38 cu. yd.		L = 56.5 ft. Volume in back-wall = 48 cu. yd.		L = 59.5 ft. Volume in back-wall = 59 cu. yd.	L = 62.5 ft. Volume in back-wall = 72 cu. yd.	L = 65.5 ft. Volume in back-wall = 87 cu. yd.	
	3 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	3 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	3 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	3 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	3 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	3 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	4 ft. 0 in. Br. St.	
10	70	79	68	77	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	10
12	90	102	89	101	88	95	92	102	.....	.....	.....	.....	.....	.....	.....	12
14	110	126	111	126	110	124	118	134	124	136	129	139	.....	.....	.....	14
16	134	149	137	148	138	151	145	167	153	169	161	176	181	185	.....	16
18	164	175	168	179	170	181	179	190	186	203	195	214	223	230	236	18
20	197	209	202	214	206	218	217	229	228	240	238	252	265	277	286	20
22	232	246	239	253	245	256	259	273	272	286	284	298	307	324	338	22
24	270	285	279	295	287	303	303	319	319	335	334	350	362	375	390	24
26	301	327	322	339	332	349	351	369	370	388	388	406	421	437	452	26
28	354	373	368	387	380	399	403	422	424	445	446	466	484	504	522	28
30	400	420	417	437	432	453	456	479	483	505	507	529	551	574	596	30
32	452	471	469	491	486	509	516	539	545	568	573	597	623	649	674	32
34	501	524	524	548	542	569	577	602	610	636	642	668	697	728	757	34
36	556	581	581	607	605	631	642	669	679	706	715	743	776	811	844	36
38	613	639	642	669	669	697	710	739	751	781	791	821	859	898	936	38
40	673	701	706	735	736	766	782	813	827	859	871	906	946	989	1,032	40
42	723	766	773	803	807	838	857	890	906	940	956	990	1,037	1,085	1,132	42
44	802	833	842	875	880	914	935	970	990	1,025	1,043	1,080	1,132	1,185	1,237	44
46	871	903	918	949	957	992	1,017	1,053	1,076	1,114	1,135	1,174	1,231	1,289	1,346	46
48	942	977	991	1,026	1,037	1,074	1,102	1,140	1,166	1,206	1,230	1,271	1,333	1,397	1,459	48
50	1,016	1,051	1,069	1,106	1,120	1,159	1,190	1,231	1,260	1,302	1,330	1,372	1,440	1,509	1,577	50

For skewed abutments, multiply respective quantities by secant of skew angle. Abutments above heavy line have a base greater than $0.45H$.

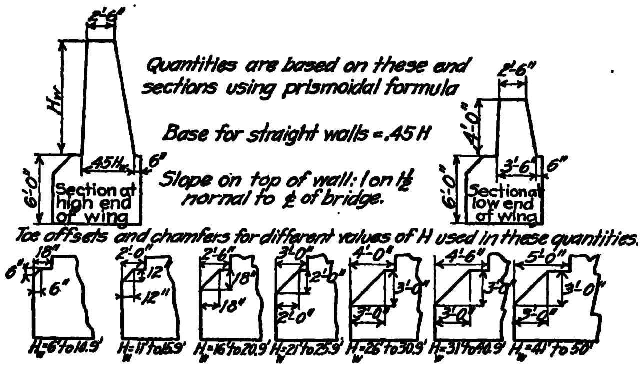


FIG. 10.

ABUTMENT FOUNDATIONS
Volumes in Cu. Yd.

Height = H, ft.	Toe offsets, ft.	in.	Chamfers, ft.	in	Foundation 6 ft. deep per lin. ft of breast[1]	Foundation per additional foot of depth per ft of breast
10	1	6	0	6	1.44	0 25
12	2	0	1	0	1.74	0 30
14	2	0	1	0	1.94	0.33
16	2	6	1	6	2.23	0.38
18	2	6	1	6	2.43	0 42
20	2	6	1	6	2.63	0 45
22	3	0	2	0	2.91	0 50
24	3	0	2	0	3 11	0.53
26	4	0	3	0	3.44	0 60
28	4	0	3	0	3.64	0 64
30	4	0	3	0	3 84	0.67
32	4	6	3	0	4.15	0.72
34	4	6	3	0	4 35	0 76
36	4	6	3	0	4.55	0.79
38	4	6	3	0	4.75	0.82
40	4	6	3	0	4.95	0.86
42	5	0	3	0	5.26	0.91
44	5	0	3	0	5.46	0 94
46	5	0	3	0	5.66	0.97
48	5	0	3	0	5.86	1.01
50	5	0	3	0	6.06	1.04

[1] No chamfer allowance in this column.

QUANTITIES IN ONE WING
Volumes in Cu. Yd.
Base at Neat Line = 0.45 H_w
Slope of top 1 on 1½

	Straight wings		
H_w, ft.	Neat concrete	Foundation 6 ft. deep	Foundation additional per ft.
6	1.7	4.3	0.8
8	4.1	8.0	1.4
10	7.7	12.7	2.2
12	12.7	19.1	3.3
14	19.2	25.3	4.3
16	27.4	33.6	5.8
18	37.6	41.2	7.0
20	50 0	49.4	8.4
22	64.6	60.2	10.4
24	81.9	70.0	12.1
26	101.8	84.5	15.1
28	124.7	95.8	17.0
30	150.7	107.6	19.1
32	180.1	124.7	22.0
34	213.0	138.0	24.3
36	249.7	152.0	26.7
38	290.3	166.6	29.2
40	335.0	187.7	32.8
42	384.1	203.8	35.6
44	437.7	220.6	38.5
46	496.1	237.9	41.4
48	559.4	255.7	44.5
50	627.8	274.2	47.7

For splayed wings, multiply respective quantities by secant of splay angle.

35. Hollow and Arch Abutments.—This type is exemplified by abutments in which arch openings are placed in the breast

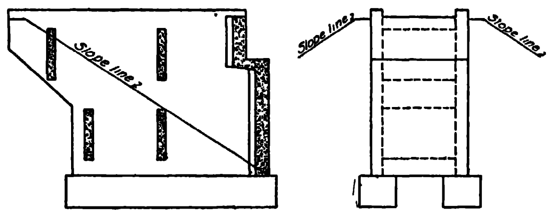


Fig. 11.—Hollow "U" abutment, Cellular Type.

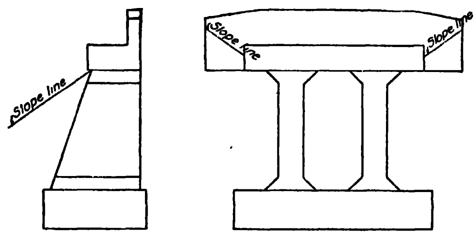


Fig. 12.—Buried abutment.

or the stem, in order to economize in the quantity of masonry. Hollow abutments are also of the cellular type. Hollow or cellu-

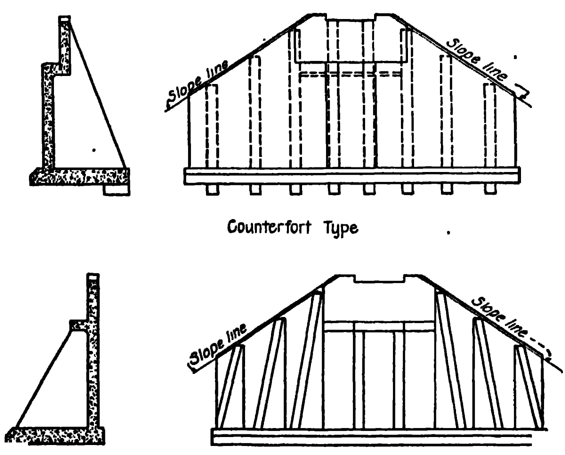


Fig. 13.—Reinforced concrete abutments.

lar abutments should be so designed as to withstand the pressure of the fill in the rear, together with the superimposed load,

including the relieving pressure caused by any fill which may be placed in front of the abutment.

Abutments of this character are indicated by Fig. 11.

36. Buried Abutments.—This term applies to abutments which are of sufficient dimensions to support the superstructure, and prevent the fill from encroaching on the bridge seat. They are not provided with wings to retain the filling material, and the fill is permitted to flow around the abutment. Such abutments are of varied detail construction, and may be plain, arch or cellular.

A representative type of this abutment is shown in Fig. 12.

37. Reinforced Concrete Abutments.—The distinguishing characteristic of this type consists in the fact that the stability

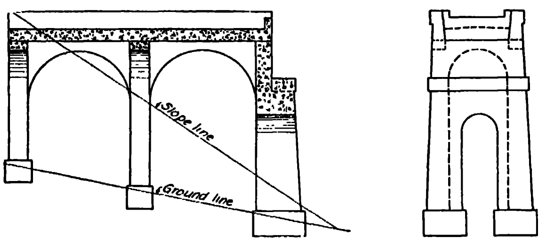


Fig. 14.—Reinforced concrete arch abutment.

as a whole is not dependent primarily on gravity action, as is the case in the ordinary abutment, but is due to the weight of the filling material. Such abutments are of the buttress or counterfort type, in which the earth thrust is resisted by counterforts or buttresses.

A typical abutment of this character appears in Fig. 13.

BASCULE BRIDGE PIERS

By Hugh E. Young

38. Counterweight Pits in General.—The piers and abutments of bascule bridges do not differ materially from those for stationary bridges, unless the tail ends of the bascule girders, or the counterweight, or both, go below the water line for any angle of opening of the bridge, in which case it becomes necessary to provide a water-tight "counterweight pit" or "tail pit." Some early examples of bascule bridges (Langebro, Copenhagen, Denmark—now replaced by a modern bridge—and Honig Bridge,

Koenigsburg, Germany) dispensed with the use of pits by employing counterweights which were suspended from the tail ends of the bascule girders by means of rods or chains and which were at all times submerged in water. Such an arrangement should, however, only be considered for temporary or semi-permanent structures owing to the rapid deterioration of the counterweight suspensions, and the danger arising from ice conditions, etc.

Counterweight pits are, as a rule, only required when the distance from high water to bridge floor is relatively small and when the counterweight is below the floor. The new Knippels Bridge at Copenhagen, built 1909, and the B. & O. C. T. R. R.'s bascule bridge over the South Branch of the Chicago River are, however, examples of bascule bridges with overhead counterweights in which tail pits are required.

Counterweight pits are also used for certain types of lift bridges, as in the Brockport lift bridge built recently over the New York Barge Canal and the Pretoria Avenue Bridge over the Rideau Canal at Ottawa, Ontario, built in 1918. Hollow piers of similar construction are used for many European swing bridges in which the lower end of the center pivot forms the plunger in a hydraulic cylinder while the pit affords the necessary room for the hydraulic machinery and the accumulators by means of which the swing span is lifted bodily before it is swung.

Counterweight pits are considered by many as being expensive, as requiring much attention, and as involving an element of risk. The question of the risk involved and of the maintenance of a counterweight pit may be answered by the fact that the City of Chicago has built 20 to 25 double leaf bridges (a total of some 50 pits) and has had little or no trouble in keeping the pits dry and clean. Counterweight pits are often the means of developing a structure of attractive appearance and one that is an asset to the community where otherwise an overhead counterweight bridge would be the only other alternative, which would be an eyesore and would be out of harmony with the present-day demand for civic beauty.

In such a case the advantages and disadvantages should be given careful consideration. The cost of a counterweight pit is not excessive when cofferdams must be used for the foundation construction in any event, and when the subsoil is bad the cost of the pit may be of less importance than the cost of the sub-piers.

The matter of keeping the pit clean—that is, free from dirt and refuse—is largely a matter of so designing the superstructure that the dirt from the roadway will not be dumped into the pit when the bridge is raised, and this condition is fulfilled in most modern bascule bridges of the trunnion type.

To keep the pit dry under ordinary conditions it is necessary to have unyielding supports, to give proper consideration in the design to the hydrostatic pressure, and to provide for the necessary water-proofing. These problems have been solved and in each individual case are certain of a satisfactory solution at the hands of an experienced bascule bridge engineer.

To guard against accidents, such as the ramming of the pit by a boat out of control, the pit should be guarded by protection piling or fenders, as required for the protection of the piers of all movable bridges. As an important part of such protection, pile clumps consisting of an effective number of piles is recommended. Besides, it is good practice to make the front wall of the pit of somewhat larger dimensions than required merely to sustain computed loads, this being, under ordinary conditions, the only part of the pit that is liable to be struck by a vessel.

39. Types and Sizes of Counterweight Pits.—To illustrate the types of construction used, counterweight pits may be classified, in general, as follows:

1. The pit is in the nature of a recess or chamber in a large massive pier. (*Example.*—Tower Bridge, London, built 1892—see Fig. 15.)

2. The pit consists of four concrete walls of approximately the same thickness—except as noted above for the front wall—and a concrete floor all resting directly on hardpan or rock. (*Example.*—Phoenix Bridge, built over the New York State Barge Canal—see Fig. 16—also 35th Street Bridge, Chicago, built 1914.)

3. The pit consists of a heavy reinforced concrete box resting on sub-piers, the walls being designed to carry and transmit the superstructure loads to the sub-piers. (*Example.*—Jefferson Avenue Bridge, Detroit[1]—see Figs. 17 and 18—also Fort Street Bridge, Detroit, and all the late bridges in Chicago—also the new railroad bridge for the Detroit, Toledo & Ironton R. R. over the Short Cut Canal at Detroit.)

[1] Jefferson Avenue and Fort Street Bridge over the River Rouge built 1922 for Wayne Co. Road Commissioners, Chicago Bascule Bridge Co., Engineers.

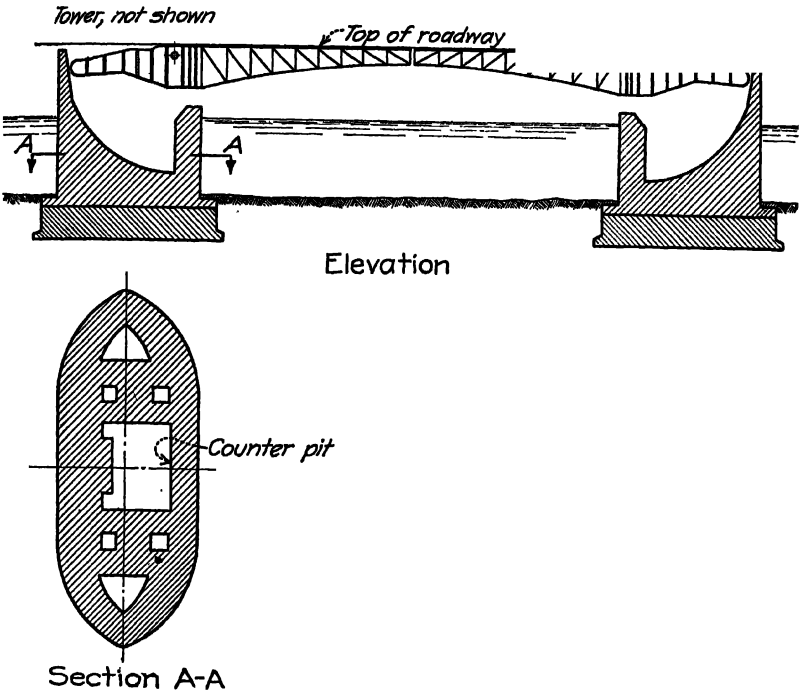


FIG. 15.—Tower Bridge, London, built 1892.

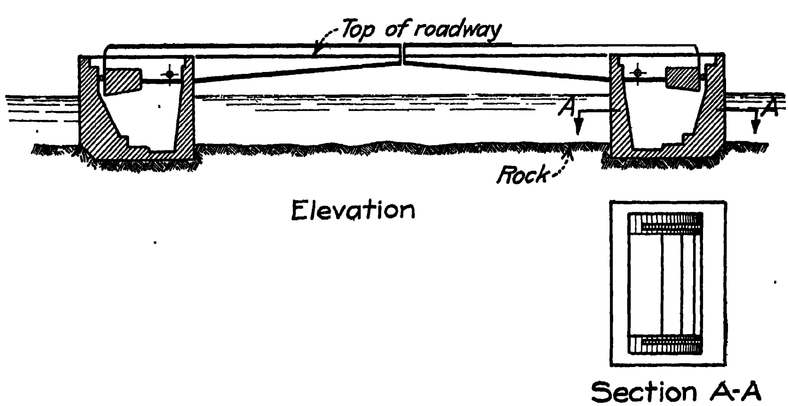


FIG. 16.—General design of Phoenix Bridge, built over the New York State Barge Canal; also 35th Street Bridge, Chicago, built 1914.

Each of the two pits for the double deck Michigan Avenue or Boulevard Link Bridge in Chicago[1] (see Fig. 22) has the following inside dimensions: width 66 ft.; length (parallel with center line of bridge) 52 ft. depth 34 ft. 5 in. below water line. These counterweight pits are probably the largest ever built.

4. The pit consists of a relatively light reinforced concrete box, suspended between two rectangular piers which may be

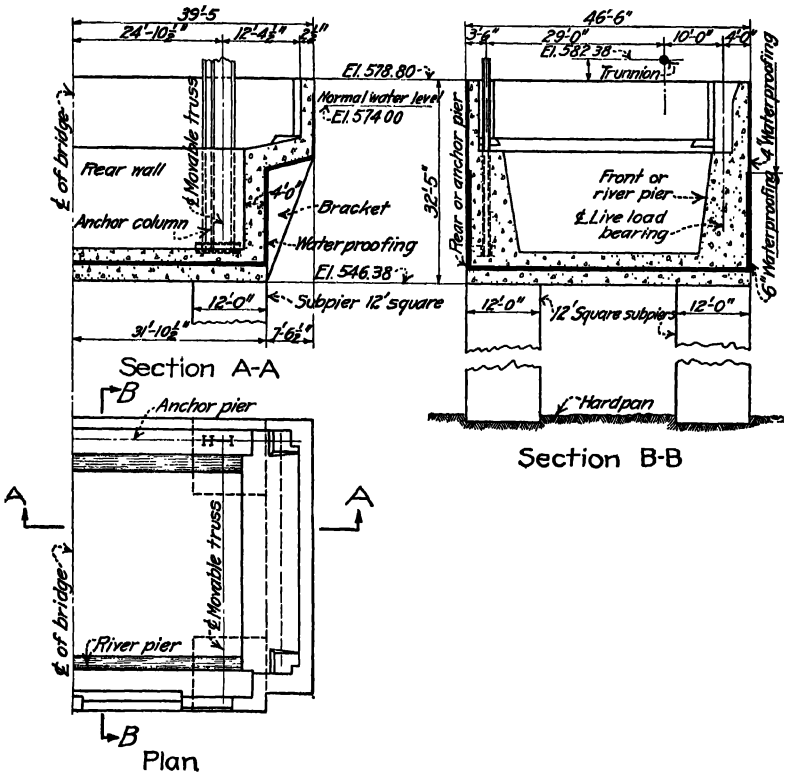


FIG. 17.—General design of Jefferson Avenue and Fort St. Bridge, Detroit; also the new railroad bridge for the Detroit, Toledo & Ironton R. R. over the Short Cut Canal at Detroit; also all the late bridges in Chicago.

founded on rock, on piles or on sub-piers and which carry the superstructure loads, while no loads whatever are carried by the pit with the exception of its own weight and the outside hydro-

[1] Michigan Avenue Bridge over Chicago River, built 1920 for Board of Local Improvements, City of Chicago, Hugh E. Young, Engineer of Bridge Design.

static pressure. (*Example.*—Erie Street Bridge, Chicago[1]—see Fig. 19.)

5. The pit consists of a light reinforced concrete box suspended on large corner piers carried down to a sub-foundation, (*Ex-*

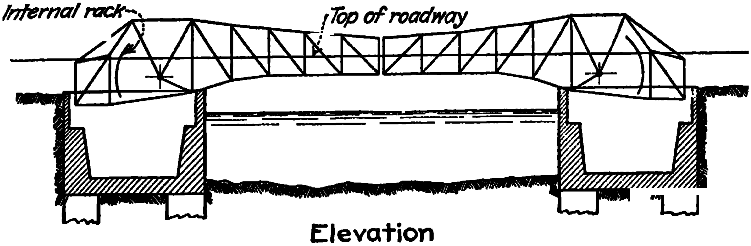


Fig. 18.—General design of Jefferson Avenue and Fort St. Bridge, Detroit; also the new railroad bridge for the Detroit, Toledo & Ironton R. R. over the Short Cut Canal at Detroit; also all the late bridges in Chicago.

ample.—Bascule Bridge over Young's Bay, Astoria, Oregon, built 1920—see Fig. 20.)

6. The pit consists of a sheet metal or cast iron box suspended between two piers. (*Example.*—Old Knippels Bridge at Copenhagen, built 1869, and now torn down—see Fig. 21.)

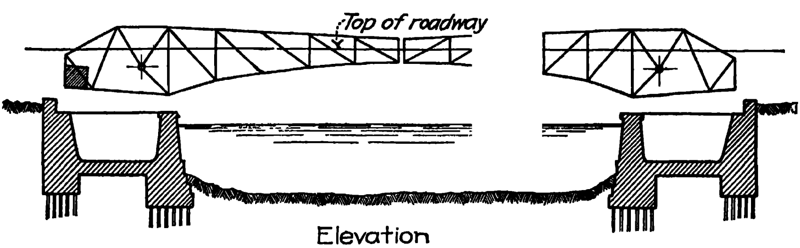


Fig. 19.—Erie Street Bridge, Chicago, built 1910.

40. Clearances.—In laying out a counterweight pit the question of clearances should be given careful consideration from the start. The clearance between the counterweight in any position and the pit walls should never be less than 6 in. and a clearance of 12 in. is to be preferred.

In case the bridge is to be erected in the open position, sufficient clearance must also be provided for riveting up the field connections in the tail ends of the bascule trusses and in the counterweight boxes. In large and important structures it is sometimes

[1] Erie Street Bridge over North Branch Chicago River, built 1910 for City of Chicago, Alexander von Babo, Engineer of Bridge Design.

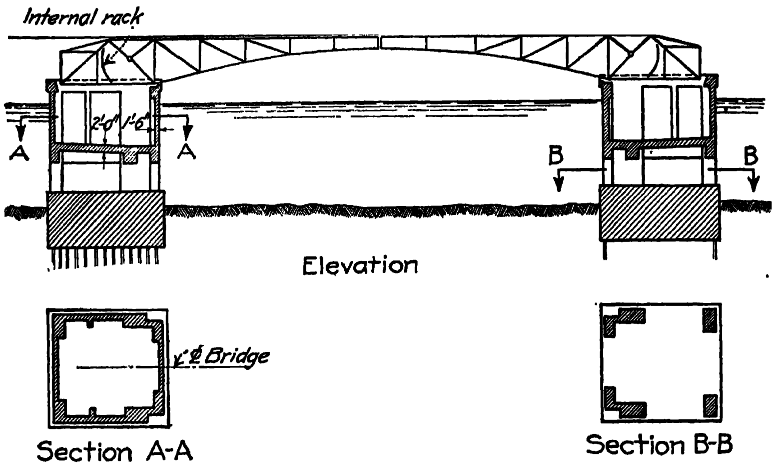


FIG. 20.—Bascule Bridge over Young's Bay, Astoria, Oregon, built 1920.

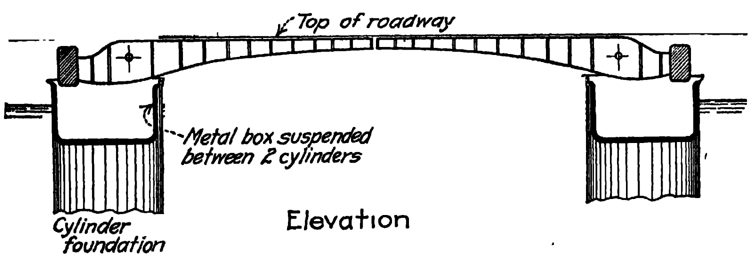


FIG. 21.—Old Knippels Bridge at Copenhagen, built 1869 (now removed).

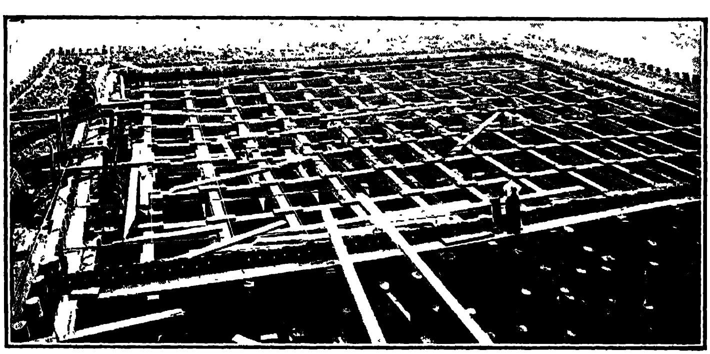


FIG. 22.—Cofferdam of the Boulevard Link Bridge in Chicago.

required that there be sufficient clearance to enable a man to save himself if the bridge should be set in motion while he is down in the pit for one purpose or another.

In general, ample clearance is desirable, (1) because it facilitates erection and the placing of the counterweight material, (2) because it will allow for a slight change in the location of the pit necessitated by a movement of the cofferdam work, and (3) because it permits of a future increase in the size of the counterweight Such an increase in the size of the counterweight has often been resorted to in order to compensate for a heavier bridge floor that may be demanded by an increase in the traffic requirements during the life of the bridge.

41. Watertightness.—The design of a counterweight pit presents no particular difficulties once the governing loading conditions have been established but in addition to having sufficient strength the pit must also be watertight, and this requires not only special provisions for waterproofing (see Art. 44) but also that the pit walls be so designed as to have sufficient rigidity to properly support and protect the waterproofing material and to preclude cracks.

This consideration may, in many cases, determine the thickness of the pit walls rather than the consideration of strength to resist computed loads. In no case should the thickness of a reinforced concrete pit wall be less than 12 in. even for a small and shallow pit.

42. Loading Conditions.

42*a*. Pit Proper.—The critical loading conditions depend on the type of the pit and must be established by a careful consideration of all the contributing factors. The maximum stress in any one part of the pit will result from the proper combination of the following loads:

(*a*) Superimposed loads due to bridge closed, no live load.
(*b*) Superimposed loads due to bridge closed, full live load
(*c*) Superimposed loads due to bridge open, no wind.
(*d*) Superimposed loads due to bridge open, maximum wind pressure.
(*e*) Dead load of pit itself.
(*f*) Water pressure from outside, pit empty.
(*g*) Water pressure from inside and no water pressure from outside.

Loads *a* and *c* will, as a rule, be identical, but in certain rolling and other patented bridges there is a horizontal translation of the superstructure load as the bridge opens and closes and the

critical position of this load must be investigated. Also a certain impact should be added to this load, varying between 0 and 50 per cent, depending on the general arrangement and, particularly, on the smoothness with which the movement takes place.

In determining loads *b*, it will, as a rule, be permissible to disregard impact and to figure only a fraction of full live load varying, say, from 50 per cent for a large and wide highway bridge to 80 per cent for a single track railroad bridge.

The loads under *f* are easily determined except the upward hydrostatic pressure on the bottom of a pit of type 2 where only a direct examination of the underlying rock can decide whether it is necessary to consider full, reduced or no hydrostatic pressure.

The loads under *g* obtain only while the pit is being tested for watertightness with the cofferdam still in place. For this condition it is customary to use unit stresses 25 to 50 per cent in excess of those used otherwise.

In addition to the above loads, which can easily be established, the pit must be strong enough to resist wave action, ice thrust,

Fig. 23.—Cofferdam of the Franklin-Orleans Street Bridge, Chicago.

and the impact from a boat drifting against the pit after having demolished the pier protection. These forces do not lend themselves to accurate calculations and are generally provided for by arbitrarily increasing the thickness of the pit walls where this seems advisable, and particularly by increasing the mass of the channel pier or the front wall of the pit.

42*b*. Foundations.—In the design of the pile foundations or sub-piers for counterweight pits the same loads as enumerated above are to be taken into account, but for this part

of the structure it is common practice to disregard live load impact and to still further reduce the percentage of the live load, except when the dead load of superstructure and pit combined is small as compared with the live load.

Fig. 24.—Counterweight pit of the Franklin-Orleans Street Bridge, Chicago.

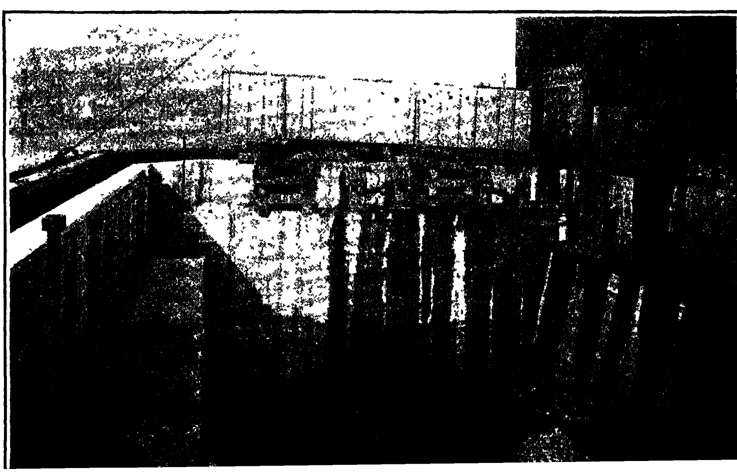


Fig. 25.—Completed counterweight pit, Franklin-Orleans Street Bridge, Chicago, Ill.

When sub-piers are used, four such piers will generally suffice, but, when the pit is unusually large, the number must be

increased, each pit of the Michigan Avenue Bridge, referred to above, for instance, being founded on 9 circular piers varying from 7½ to 12 ft. in diameter.

An increase in the number of sub-piers may also be necessary where utilities, tunnels or other obstructions interfere with the regular spacing of the sub-piers—as was the case in the south pit of the Franklin-Orleans Bridge in Chicago[1] (Figs. 23, 24, and 25) which is supported on five circular piers varying from 6½ to 10½ ft. in diameter. (Two additional sub-piers, 5 ft. in diameter, were placed under the house foundations.)

The sub-piers are generally circular and vary in size from 6 to 14 ft. in diameter, but in a series of bascule bridges recently constructed over the River Rouge at Detroit it was found necessary to use 12 × 12 ft. square sub-piers, these piers being sunk by the pneumatic process and through very bad bottom (see Fig. 26).

43. Unit Stresses.—The unit stresses employed in the design of counterweight pits should conform with standard practice.

44. Materials.—The material most commonly used, and, under ordinary conditions, most suitable for counterweight pits, is reinforced concrete in rather heavy sections and so reinforced as to preclude cracks. A veneer of cut stone is sometimes used and is very desirable in salt water. In the modern Chicago and Detroit bridges a distinction is made between the pit proper, which is built entirely of concrete and extends only a few feet above the water line, and the enclosure walls built on top of the pit and extending up to the bridge floor. These walls are of stone masonry—either granite or limestone—with brick backing. In this manner, utility, economy, and appearance are duly served.

Steel or cast iron boxes and suspended reinforced concrete pits with thin walls have been used but are not considered standard practice. The pit is a part of the substructure and should be designed so as to possess the same degree of permanence, without maintenance, as is requisite in the foundations for any permanent structure.

44*a*. Concrete Mixture.—The concrete should be a 1:3:5 mixture or, when the walls are rather thin, a 1:2:4 mixture to which should be added about 10 lb. of hydrated lime for each bag (94 lb.) of cement.

[1] Franklin-Orleans Bridge, over Chicago River, built 1920 for City of Chicago, Hugh E. Young, Engineer of Bridge Design.

44b. Waterproofing.—The waterproofing of a counterweight pit presents about the same problems as the waterproofing of a deep basement or other pit and is accomplished in a similar manner.

Two methods have been employed: (1) integral waterproofing, and (2) waterproofing by means of an unbroken layer or seam of dense and impermeable cement mortar.

Integral waterproofing compounds have been used with success in many cases but recent laboratory tests and research give promise that it will soon be possible—when the available aggregates are known—to establish in advance the particular concrete mixture which, without the use of a waterproofing compound, will produce impermeable concrete under such conditions as obtain in a counterweight pit.

The use of a waterproofing membrane is not favored, this system of waterproofing being, in fact, more suitable for structures subject to water pressure from the inside.

When the water-tightness of the concrete is not to be relied on, excellent results can be obtained by means of a continuous layer of cement mortar, 1:2 mix, varying in thickness between 4 and 6 in. according to the size of the pit and the pressure (see Fig. 17).

45. Construction.—Construction methods vary greatly with the size and the design of the pit and with local conditions. Only a few points of special interest and importance will be mentioned.

When piles are used in the foundation, they are generally driven before the cofferdams. The cofferdam is then built and as the water is pumped out, bracing is put in, and the piles are cut off as necessary to give room for the bracing.

When sub-piers are used and conditions are favorable for building them in open wells—as in Chicago—the cofferdams are as a rule completed and pumped out before the wells are sunk, while the reverse is found more convenient when the sub-piers are sunk by the pneumatic process as at Detroit (see Figs. 26 and 27).

The particular difficulties that are met with in the construction of a counterweight pit arise from the box-like shape of the structure, from the necessity of watertightness and from the fact that the dimensions of the cofferdam are generally large, necessitating heavy cofferdam bracing which must at all times be supported without interfering with the placing of the concrete

bottom and walls (see Figs. 22 and 23). To overcome this difficulty the following procedure has been carried out in many of the Chicago bridges.

Four to six piles are driven on the longitudinal or transverse center line of the pit for the support of the cofferdam bracing.

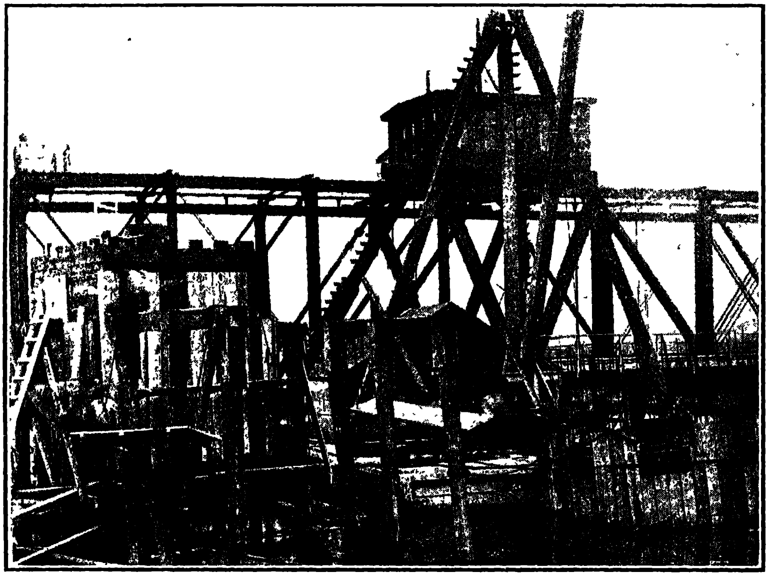


FIG. 26.—Sub-piers of the Jefferson Ave. Bridge, Detroit, Michigan.

The bottom of the pit is poured in two layers, separated by a 4 to 6 in. layer of cement mortar.

While the bottom layer is being poured, the piles are surrounded, where they pass through the floor, by a wooden box, shaped as a truncated pyramid (see Fig. 24).

When the concrete has set, the pile is cut off 3 or 4 ft. above the concrete and supported on a horizontal timber, which, in turn, is supported on two blocks resting on the concrete

The pile stump is then cut off, the box removed, and the hole filled with concrete—the tapered form of the concrete plug insuring tightness under outside water pressure.

The mortar layer is then placed and the upper half of the floor slab is poured—the blocks or stub-posts supporting the pile being surrounded by wooden boxes as before.

When the concrete in the second layer has set, the blocking is shifted, the wooden boxes removed, and the holes filled with concrete.

It may be advisable to do the work in three operations instead of in two so that the breaks in the bottom layer, the mortar layer, and the top layer

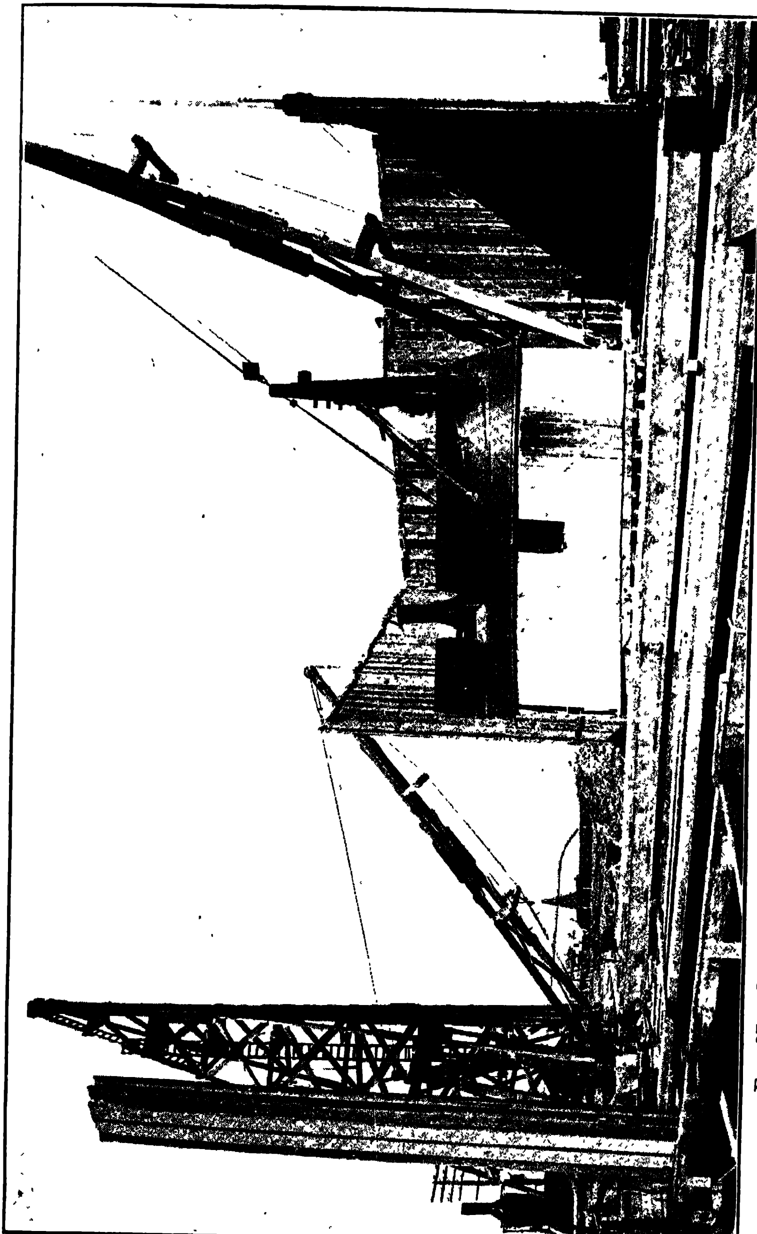


Fig. 27.—Cofferdam of the Jefferson Ave. Bridge, Detroit, Mich. (sub-piers have been driven).

will be offset one in relation to the other, but experience does not seem to indicate that this is necessary.

Watertight work is essential and to this end concreting should proceed as rapidly as possible. Each layer of the floor should be poured in one operation and no more than 3 hr. must elapse between the placing of two batches coming in contact with each other.

The walls from the pit floor to the water line should also be built in one continuous operation except for such short interruptions as may be necessary in order to allow for the removal and reframing of the cofferdam bracing.

Particular attention must be paid to the joint between the floor and the walls. The joint should be trenched. The bottom reinforcement should be bent and continued well up into the wall (see Fig. 24) and, before the concreting of the walls is commenced, the surface of the joint must be roughened, cleansed of scum, laitance, and foreign material and then wetted down and slushed with neat Portland cement.

The waterproofing course of cement mortar in the pit bottom must be placed in one continuous operation. The waterproofing course in the walls must be carried up with the concrete and this is conveniently done by means of forms consisting of steel plates, about 12 in. wide, to which are riveted vertical angles which bear against the inside of the outside wall of the wooden forms. The thickness of the layer, 4 or 6 in., will determine the size of the angles to be used. These steel forms are easily pulled up as the concreting progresses and are held in place by the pressure of the concrete which is deposited so as always to be slightly ahead of the mortar layer.

46. Testing.—Before the cofferdam is removed, the pit may be tested for watertightness by filling the space between the cofferdam and the pit with water after the concrete in the pit has had sufficient time to set.

If leaks appear, the water should again be pumped out and the leaks—if necessary—traced to the outside of the pit by filling the pit with water. After carefully stopping up the leaks the pit should be pumped out and the cofferdam again flooded. These operations must be repeated until a satisfactory job is obtained. In practice, however, the pit is generally found to be tight from the beginning and with proper design, proper materials, and proper construction methods, it is now possible to insure a watertight job without the necessity of repeated tests.

CYLINDER AND PIVOT PIERS

By Philip George Lang, Jr.

47. Cylinder Piers.—Cylinder piers are formed of two or more cylindrical columns, usually consisting of a steel shell filled with concrete. These cylinders must be properly strutted and tied together to form a pier unit, and to provide the necessary lateral stability.

48. Pivot Piers.—This designation is adopted for piers supporting the center of a swing draw bridge. The horizontal section of such a pier is usually circular, of sufficient diameter to properly support and anchor the rack used in turning the bridge, and, in the case of a rim-bearing bridge, the track. Pivot piers are sometimes made octagonal in horizontal cross-section.

Incidentally, it may be stated that, in the case of a draw span, both the pivot pier and the end piers or abutments must be protected by timber fenders, to prevent damage to the substructure by water-borne traffic.

SECTION 8

LEGAL PROVISIONS REGARDING FOUNDATIONS AND FOOTINGS[1]

By W. R. Matheny
Of the Chicago Bar

1. The Engineer and the Law.—The Engineer from the nature of his work deals almost entirely with the future, in preparing plans and estimates of what is to be, while the Lawyer deals primarily with things that have passed, a cause of action not ordinarily arising until there has been a breach of a right. The law prescribing a remedy for the breach of a right must be determined, not from a calculation based on scientific principles but from the decisions made by courts on similar or analogous facts.

The Engineer has given the matters of law little consideration. He ordinarily believes that the law consists of rules enacted by legislative bodies. In this he is partially correct. But the great mass of the law under which we live is custom, solidified by usage, modified, affirmed or repealed by our law-making bodies, and interpreted, explained and qualified by our judicial tribunals.

A competent Engineer may be said to know his branch of the art. Yet no lawyer could claim to know the law, a compact working law library today consisting of some 15,000 volumes. And Congress, 48 state legislatures, 49 supreme courts, as many courts of appellate jurisdiction, all in this country alone, are making, amending, repealing and modifying the law each day.

2. Modern Law.—The law today consists of:

The Federal constitution
Treaties with other nations
Federal statutes
The state constitution
State statutes
Municipal laws or ordinances
The common law

[1] The field of the law is so vast that this chapter undertakes merely to review a few principles of the Law of Real Property concerning foundations and footings. The introductory paragraphs attempt an explanation of the sources of the law together with the present-day method for its application in civil matters.

all as interpreted by:

U. S. Supreme Court
U. S. Circuit Court of Appeals
State Supreme Courts
State Appellate Courts
Regulatory Commissions

Constitutions, statutes and ordinances, the enacted law, may be found in books usually official in character. The decisions of courts and commissions are reported both officially and by private interests, being indexed in encyclopedia and digests, and cross-referenced in various ways. The common law today may be found in various text-books and in the decisions in reported cases.

It has been the policy and rule of our courts to follow the action taken in earlier cases. "Let the decision stand" has been the motto for a thousand years. Legislative modification is a slow process, and often illy guided. This results in a lack of modernness bemoaned by many. Yet this system, in use only among the English-speaking nations is far preferable to any other yet devised. When each case under a set of fixed and rigid rules is tried by a human judge who decides each case by comparing its facts with a rigid and definite rule, without regard for precedent, uniformity and consistency are necessarily lost. On identical facts one man may win and another lose. And the basic principle of modern government, equality before the law, is disregarded.

3. The Function of the Civil Law.—The Engineer should know that the Law, the mass of customs, statutes and decisions, has two functions: (1) to establish rights and duties; and (2) to protect those rights, enforce those duties, and provide compensation for their breach. The law grants to each person the right to live, to be free, to hold property, and to contract with other persons. The law imposes a duty on all persons to respect those rights. And the law supplies a remedy for their infringement, including in some cases measures for the prevention of infringement. Without these rights, duties and remedies, civilized society could not exist.

4. Subdivisions of the Law.—The great body of the Law is divided for convenience into various branches. No exact division can be made, as each branch merges into the other branches. Each of the subdivisions affects the engineer on foundation work directly, indirectly or remotely, generally in accordance with the following tabulation:

Law of Rights and Duties

Direct	*Indirect*	*Remote*
Real Property	Criminal law	Constitutional law
Contracts	Torts	Wills and estates
Agency	Corporations	Admiralty
Negotiable instruments	Partnership	Patents and Copyrights
Workmen's compensation	Sales	Bankruptcy
Negligency	Suretyship & guaranty	
	Bailments	
	Insurance	
	Common carriers	

Law of Remedies

Direct	*Indirect*	*Remote*
	Law pleading and practice	Probate procedure
	Equity pleading and practice	
	Damages	
	Evidence	

5. Rights and Duties Concerning Land.

5*a*. Definitions.—*Land* is the ground, soil, or surface of the earth and includes everything erected upon its surface or which is buried beneath it—*Tiedeman on Real Property.*

Land includes whatever is parcel of the terrestrial globe or is definitely affixed to such parcel—*Tiffany on Real Property.*

The term *Land* includes buildings, trees, growing things, fences, bridges, rocks, mines, minerals, fossils and all natural products of the soil—*Burdick on Real Property.*

The term *Land* also includes standing water, and percolations beneath the surface. Running waters are not included in the term, whether on or beneath the surface, the owner of the land having only the right to the use of such streams.

Property includes those things and rights which are the objects of ownership.

Ownership is the right to exclusive possession and the right to use the thing or right owned practically without limitation.

Ownership includes the right to the control and enjoyment of that which is owned and the right to improve or destroy it, subject to the rules of the law—*Walsh on Real Property.*

5*b*. Reasonableness of Use.—The rights which ownership or dominion confer upon the owner are generally unlimited. The State reserves always the right to impose taxes, and the right to take the ownership of property for public

purposes with compensation under the doctrine of Eminent Domain. There is always, however, a further limitation; the use of land must be reasonable, with respect to the rights of others. This rule of the law is expressed in the maxim "sic utere tuo ut alienum non laedas," which means that one must use his land so that injury will not result to the land of another. This rule does not restrict reasonable and prudent use and enjoyment.

The test of reasonableness is whether the use is such a use as an ordinarily prudent man would make of his own premises considering the importance of the use to the owner as well as the extent of the damage which might result to the premises of his neighbor. If the owner uses ordinary care to prevent unnecessary injury, the fact that the injury would inevitably be produced does not necessarily make the use negligent.

Unskillful use will usually give rise to a right for damages should damage result.

5c. The Right to Lateral Support.—The modern rule as to lateral support was established in England in the case of *Angus vs. Dalton,* first reported in 1877 (L. R. 3, Q. B. D .85), reviewed on appeal in 1878 (4 Q. B. D. 163) and finally decided in 1881 (6 App. Ca. 740). Until that time no definite rule can be found.

The rules in this country as set forth below have been established by *Shrieve vs. Stokes* (Kentucky, 1848) 47 Ky. 453, 48 Am. D. 401, *Black vs. Haseltine* (Indiana 1892) 29 N. E. 937, and *Obert vs. Dunn* (Missouri App. 1897) 41 S. W. 901.

An owner of land is entitled to have his land supported and protected in its natural condition by the land of an adjoining proprietor. This right of lateral support is not merely an easement but is a right of natural property incident to ownership of the soil. This right exists in every state, independent of statute.

The right to lateral support pertains only to land and does not extend to buildings which increase the lateral pressure. A mere presence of buildings is not a defense when such buildings do not contribute to the injury.

A right to the support for buildings may be acquired by grant, express or implied. Modern decisions do not permit the acquiring of this right by prescription, or long usage, in this country.

The right to lateral support is confined to the adjoining premises, but injury to land caused by excavation is not so limited.

It is the duty of an owner to excavate in an ordinarily skillful and careful manner where buildings on adjoining land are concerned. There is no duty to excavate by piecemeal or to build a wall in sections. It is immaterial that an adjoining building is poorly constructed.

In excavating the owner must protect his neighbor's soil from falling. Actual injury must have been caused to permit recovery.

The excavating owner should notify an adjoining owner of his intention to excavate under the general maxim of 'sic utere.' Notice is necessary where an excavation goes below the foundation of an adjoining building. No formal notice is required; it is sufficient if full knowledge is imparted to the adjoining owner.

If the adjoining owner neglects to take proper precautions after notice, the excavating owner is not liable unless he excavates in a negligent and imprudent manner. A change in the plan of excavation after notice will make the excavator liable. Neglect of an adjoining owner to take precautions will allow the excavator to protect buildings at the owner's expense but after notice the excavator is under no duty to provide such protection.

A license, or permit, is necessary to permit the excavating owner to enter adjoining premises for the purpose of providing protection. This matter has been the subject of legislation by some states and cities, the laws of which should be consulted.

A promise to furnish protection will ordinarily make the excavator liable.

Where a right to support of land or buildings has been acquired by an adjoining owner, he is under no duty to protect his property, the excavating owner being liable for any damage which may result.

An excavating contractor is ordinarily liable. If the owner reserves any right of direction over the contractor, the owner would also be liable.

One who purchases land, after an excavation has caused damage to adjoining land, is not liable. The liability rests upon the owner rather than upon the land.

One who agrees to protect the buildings of another is liable even though such promise is without consideration.

Where statutes or ordinances prescribe the depth to which excavation can be made, an owner has been held liable if an independent contractor exceeds such depth.

As in other cases consent to the act bars a recovery of damages.

In determining liability resulting from excavation, the custom of practice in such work is used to determine whether or not there has been negligence, and as to whether reasonable skill and care have been exercised.

An injunction will be granted to prevent the removal of soil endangering the stability of the soil of an adjoining land owner—*High, Law of Injunctions.*

5*d*. Subjacent Support.—Where the property of one owner is located above that of another owner there is a duty upon the owner of the lower stratum to support the upper stratum.

In the event of subsidence caused by acts of the lower owner, he is liable.

Each subsidence constitutes a new and different injury.

5*e*. Raising Level.—An owner who raises the level of his land must provide reasonable protection to prevent damage to adjoining land.

5*f*. Encroachments.—An owner whose property encroaches on that of another may be required by an order of a court to remove such encroachments. There is no natural right to the support for land by underground water.

5*g*. Blasting.—An owner is liable for damages to the property of another caused by blasting, even though proper precautions were used. If blasting is conducted without the usual customary safeguards, the blasting may be stopped by the courts.

5*h*. Independent Contractors.—Where work is being prosecuted by a contractor over whom no element of control is retained by the owner, the contractor is liable for damages resulting from a prosecution of the construction work.

5*i*. Light and Air.—In this country there is no right to receive light and air over the land of another. The rule is to the contrary in England. Such a right may be acquired by an express grant. Pollution of air may give rise to a right for damages as a nuisance clause of the 'sic utere' theory rather than as an interference with light and air.

5*j*. Trees.—Trees belong to the land on which they stand. Trees actually on the boundary line are owned jointly.

5*k*. Water.—An owner of land has the exclusive right to the use and enjoyment of standing water, and to water which percolates beneath the surface.

A riparian owner who owns land adjacent to flowing water, either on or below the surface, has a right only to the use of such water consistent with the similar rights of other riparian owners. To constitute a stream so that riparian rights may be acquired there must be a regular flow of water in a well defined manner.

The setting back of water by a lower owner to the land of an upper owner is an unreasonable use.

5*l*. Highways, Streets and Alleys.—The ownership of roads and highways in this country is usually in the adjoining land owner. Streets and alleys dedicated to the public use are usually the property of the municipality. This is true in most of the western states. In the older states of the east the public easement theory is more generally followed. Local statutes and rules should be consulted.

5*m*. Building Codes.—In many municipalities and in some states there have been specific enactments covering the use of real property. In the absence of such rules the general principles of common law govern.

6. Remedies.—The Engineer will seldom be required or expected to know the remedies available for the breach of a duty or the infringement of a right. The remedies available are in two classes, the first resulting in damages or a money settlement, and the second resulting in a restraining order issued by a court. Damages cannot be secured until damage has resulted. Restraining orders or injunctions may be issued when damages will apparently result from an intended act or more exactly when injury will result from acts for which money damages will not be a sufficient compensation.

7. Conclusion.—The Engineer should know that, while everyone is charged with a knowledge of the whole of the law, the legal system has become so intricate that the services of a specialist, one trained in the law, are necessary for the safeguarding of the interests of those whose rights are threatened or who have interfered with the rights of others. The Engineer thinks that the Lawyer is responsible for the present complexity. Perhaps the Engineer is right. But he must meet the situation as he finds it.

APPENDIX *A*

Extracts from Progress Reports of Special Committee of American Society of Civil Engineers to Codify Present Practice on the Bearing Value of Soils for Foundations

DEFINITIONS OF SOILS

Your committee has given further consideration to definitions for soils that will combine common and practical ideas with controlling physical factors. The difficulty is emphasized, however, through the frequent confusion of the physical state of the soil material with the soil as a mass. For example: "Quicksand" is a physical state of water and granular material, rather than a type of soil.

A general discussion of soils met with in construction work by members of the Society might develop further suggestions for definitions. For this reason some common definitions of soils are presented, as follows:

Alluvium.—The finer deposit of earth, sand, gravel, and other transported material, usually occupying the lower parts of valleys and great rivers, which has been washed away and thrown down by rivers, floods, or other causes, on land not permanently submerged.

Bog.—A quagmire covered with grass or other plants. It is defined by marsh and morass, but differs from a marsh as a part from the whole. Wet grounds are either bogs, which are the softest and too soft to bear a man; marshes or fens, which are less soft but very wet; or swamps, which are soft, spongy land on the surface, but sustain man or beast, and are often mowed. A little elevated spot or clump of earth in marshes and swamps, piled with roots and grass—this is a common use of the word in New England.

Clay.—A general name for cohesive soils. The name of certain substances which are mixtures of silex and alumina, sometimes with lime, magnesia, alkali, and metallic oxides. A species of soil which is firmly coherent, weighty, compact, and hard when dry, but stiff, viscid, and ductile when moist, and smooth to the

touch, absorbs water readily but not greedily, diffusible in water and, when mixed, not readily subsiding in it.

Clay is also defined as the material resulting from the decomposition and consequent hydration of the feldspathic rocks, especially granite and gneiss, and of the crystalline rocks in general. As thus formed, it almost always contains more or less sand, or silicious material, mechanically intermixed. After this has been separated, the clay itself is found to consist of a hydrated silicate of alumina, but it is not yet positively determined that there is one definite combination of this kind constituting the essential basis of all the substances to which the name clay is applied. All clays contain hygroscopic water which may be expelled by heating to 212 deg. F., but they also contain water in chemical combination, and when this is driven off by ignition the clay loses its plasticity and shrinks in volume, neither of which can be restored by the addition of water. The lime and other impurities present in ordinary clay render it to a certain extent fusible. The purer varieties are refractory and are known as fire-clay. The plasticity of clay is of great importance, as without this quality it could not be easily worked into the various shapes for which it is used. On what condition it depends has not as yet been clearly determined. Clay is any mixture of silica and alumina in a finely pulverized condition; a mixture of granular materials and a colloid.

Clay is also defined as mixtures of minerals of which the representative members are silicate of aluminum, iron, the alkalis, or the alkaline soils. The hydrated aluminum silicate, kaolin, is the most characteristic of these. Some feldspar is usually present. The grains of these minerals may show crystal faces (especially in the case of kaolins), but more commonly they are of irregular shapes; upon most of these grains is an enveloping colloid coating. This is mainly of silicate constitution, but may consist partly of organic colloids, of iron, manganese, and aluminum hydroxides, and of hydrated silicic acid. Quartz grains, which are generally present, do not have the colloidal coating, or have it in much less degree. Almost any mineral may be present in clay, and modify the properties somewhat. The combination of granular materials and colloids is in such proportion that when reduced to sufficiently fine size (by crushing, sifting, washing, or other means) and properly moistened with a proper quantity of water, plasticity is developed. If the colloid matter is in excess the clay is consid-

ered to be very plastic, fat, or sticky, but if the granular material is in excess, it is called sandy, weak, or non-plastic.

The colloid matters in clay are non-crystalline, hydrated, gelatinous, aluminum silicates, organic colloid, gelatinous silicic acid, and hydrated ferric oxide. Rarely, aluminum hydrate may also be present.

Detritus.—A mass of disintegrated rock material, loose or uncompacted, worn and broken off from larger solid bodies, either waterworn or angular, and reduced by attrition to relatively small portions, as diluvial detritus. The term is especially applicable to a material which would be a breccia, or conglomerate, if consolidated into rock. When the portions are large, the word débris is used. More comprehensively, any broken or comminuted material worn away from a mass by attrition; any aggregate of loosened fragments or particles.

Diluvium.—A superficial deposit of sand, loam, gravel, pebbles, or other coarse detrital material wherever found, caused by the deluge or ancient currents of water.

Drift or Glacial Drift.—A heap of loose detrital material, fragments of rocks, boulders, sand, gravel, or clay, or other soil driven together, or a mixture of two or more of these deposits, resting on the surface of the bedrock.

Dust.—Fine, dry particles of soil, or other matter, so reduced to powder or attenuated that it may be raised or wafted by the wind; powder; fine soil.

Earth.—The particles which compose the mass of the terraqueous globe, but more particularly the particles which form the fine mold on the surface of the globe; or any indefinite mass or any portion of that matter.

Glacial Drift.—See *Drift.*

Gravel.—Small stones or fragments of stone or very small pebbles larger than the particles of sand, but often mixed with them.

Gravel may be caused to cohere by infiltrated calcareous or silicious matter, or by the effect of such infiltration combined with that of pressure, and is sometimes called natural concrete, and indurated gravel, conglomerates, and breccia.

Grit.—Angular, rough, hard particles of sand or gravel in a loose form. The term is also sometimes applied to this material in a combined, solidified form; for example, certain classes of sandstone from which grindstones are made.

Ground, or Filled Ground (Made Land).—The surface of land, or upper part of the earth, without reference to the materials which compose it. Ground is applied to soil, indifferently, but it is never applied to the whole mass of the earth, nor any portion of it when removed. We never say a shovelful of ground.

Hardpan or Pan.—The hard stratum of consolidated soil underlying the surface soil; loess of 50 to 75 per cent silt and up to 15 per cent clay.

Loam.—A natural mixture of sand and clay with oxide of iron; a species of soil of different colors, whitish, brown, or yellow, readily diffusible in water. A clay soil containing more or less of carbonate of lime, and consequently effervescing with an acid.

Marsh.—A tract of low land, usually or occasionally covered with water, or very wet and miry, and overgrown with coarse grass or detached clumps of sedge; a fen. It differs from a swamp which is merely moist or spongy land, often producing valuable crops of grass. Low land occasionally overflowed by the tides is called salt marsh or tidal marsh.

Mold or Mould.—A fine soft soil, such as constitutes garden or vegetable mold.

Muck.—The term muck as commonly used by engineers and contractors generally means any excavated material removed or to be removed from an excavation. Hence, the allied term, "muckers" applied to laborers who handle broken rock, as well as earth or other excavated material.

A wet slimy mass of decaying or putrified vegetable matter; swamp muck; imperfect peat; the less compact variety of peat, especially the paring or tuff underlaying peat.

Mud.—Moist and soft soil of any kind, whether produced by rains on the earth's surface or by ejections from springs and volcanoes or by sediment from turbid waters; such material as is found in marshes and swamps, in the beds of rivers and ponds, or in highways after rain, easily yield to pressure.

Peat.—A brown soil of vegetable origin consisting of partly decomposed roots and fibers, more or less saturated with water. It is found in every stage of decomposition, from the natural wood to the completely black vegetable mold. It is produced under various conditions of climate and topography, and is of considerable importance in certain regions as fuel. Peat is very spongy, and contains a large quantity of water near the surface;

the deeper down it is taken, the more compact it is. It is formed of vegetable matter undergoing decay and in some respects it is the modern representative of the coal of the earlier geological epoch.

Pebbles.—Roundish stone of any kind, from the size of a nut to that of a man's head.

Quagmire.—A soft, wet, swampy land, which has a surface firm enough to bear a person, but which shakes or yields under the feet.

Quicksand.—Sand easily moved or readily yielding to pressure; loose sand, abounding with water, such as a movable sand-bank in a sea, lake, or river; a large mass of loose or moving sand mixed with water formed on many sea coasts, at the mouths and in the channels of rivers, etc.; sand supersaturated with water temporarily, and when under pressure acting as a fluid.

Rock.—A large mass of stony matter, either bedded in the earth or resting on its surface.

Rock-flour.—Microscopic sand, or rock pulverized to a degree of fineness resembling powder or dust.

Sand.—Any mass or collection of fine particles of stone, particularly fine particles of silicious stone, but not strictly reduced to powder or dust; dune sand.

Shale.—A fine-grained, indurated, clayey rock having a slatey structure.

Silt.—Fluvial sediment of mud or fine soil deposited from running or standing water.

Soil.—The unconsolidated veneer covering the rock crust of the earth. The upper stratum of the earth; the mold, or that compound substance which furnishes nutriment to plants, or which is particularly adapted to support and nourish them.

Till.—See *Drift.*

MINERALOGICAL COMPOSITION

Your Committee has given further consideration to the influence of the mineral composition of soils. The failure of certain soils by decomposition or crushing of particles, has led to a fruitful field for study in the relation of the mineral composition to the physical factors. In this connection "sedentary" soils frequently retain the original minerals unchanged, whereas in "transported" soils the minerals present are those that most tenaciously resist wearing and weathering.

Mineralogically, the soil varies with its chemical composition; it may be calcareous, alkaline, ferruginous, inicallous, silicious, etc. The presence of oxidizable sulphides may cause heating and disintegration of the soil when exposed to the air. According to Dr. Donald F. MacDonald this happened locally in the Gaillard Cut, Canal Zone, Panama. It sometimes happens in coal and culm piles.

The leaching out of soluble sulphates or other salts may cause clays to disintegrate rapidly and slide. Calcareous material, if present, may also be dissolved out of clays, causing them to lose cohesion.

It is hoped that those who are mineralogically expert will present constructive criticism, or their experience with soil failures that can be attributed to the mineral nature of the soil.

The following is a synopsis of a study, by Dr. G. N. Coffey, of twenty-five surface soils from the Coastal Plain, Piedmont Plateau, and Limestone Sections of the United States, as found in Bull. **85**, U. S. Department of Agriculture.

CHARACTERISTICS OF SURFACE SOILS

(*A*) Arid soils have a large percentage of minerals other than quartz.

(*B*) Humid soils, with the exception of orthoclase and microcline, have less abundance of feldspars.

(*C*) The influence of topography on the surface soil is often very marked. In mountainous regions erosion allows only a thin mantle of soil to accumulate. The minerals show a very slight alteration indicating that weathering has not been acting for a very long period. In the plateau regions the surface is not broken, giving rise to less erosion, but more advanced decomposition.

(*D*) Limestone soils have little variations in minerals, and such as are present occur in very small particles.

(*E*) In the unconsolidated water-laid deposits of the Coastal Plain a high percentage of quartz is abundant.

(*F*) Soils formed from glacial material are characterized by a relatively large percentage of minerals other than quartz, especially in the sands. The grains are only slightly rounded.

(*G*) In loessial soils about 75 per cent of the soil mass consists of silt. The grains are mostly angular, with some fairly well rounded.

(*H*) The total number of minerals found in the twenty-five surface soils is 34, the average number present in a sample being a fraction more than 13.

(*J*) There appears to be a considerable variation of mineralogical composition. Soils usually have a greater variation than rocks, since they are the dispersed products of rocks through degeneration and decomposition.

(*K*) The characteristics of the different minerals are as follows: Quartz is the most abundant mineral occurring in every sample. Quantitatively, quartz constitutes from 50 to 95 per cent of the surface soil, the average being 83 per cent, but it constitutes less of silt than of sand, indicating that a smaller percentage would be found in clay. The quantity of quartz is greatest in surface soils derived from rocks that have undergone the most attrition and decomposition.

Quartz occurs in largest quantities in the southeastern part of the United States.

Epidote is the next most common mineral, followed by hornblende and the two feldspars—orthoclase and microcline. The latter were not found in the reddish soils.

Of all the varieties of minerals, feldspar occurs next in abundance to quartz. The plagioclase feldspars—labradorite, andesine, oligoclase, and albite—do not appear where the soil is subjected to the greatest leaching and attrition.

The micas—biotite and muscovite—occur rather frequently. Chlorite, zircon, tourmaline, and rutile are all common minerals, but are seldom abundant, the three latter being hard-weather resisting minerals.

The apparent reason for the small occurrence of the iron minerals—magnetite and hematite—in surface soils is due to the fact that organic matter attacks iron very readily. Iron is commonly found in rocks.

(*L*) The mineralogical determinations were applied solely to the twenty-five surface soils from a composite of ten samples. Since the surface soils in the southeastern portion of the United States are more sandy than the subsoils, a large percentage of quartz might be expected in the latter. It is also probable that the larger percentage of organic matter in the surface soils would cause more leaching to take place, and that the subsoil would not only show a smaller percentage of quartz but also a greater variety of minerals.

A greater percentage of minerals is found in the north, due to the glacial action of grinding down of rocks containing various materials.

REVISED SOIL CLASSIFICATION

Your Committee submits a revision of the proposed classification of soils to which has been added a further sub-division termed "colloids," for the determination of which a centrifuge of very high rotating speed is necessary (about 40,000 r.p.m.). The Committee is not at present in a position to enter into details as to this class of material, but tension experiments conducted upon mixtures of the "colloids" and Ottawa sands have shown such surprisingly high strength as to indicate its great importance. In fact, it now appears that this element alone, or together with water, may account in a large measure for the cohesiveness of soils. The subject is to be further followed up by the Committee, and reported on later.

"Structure" has been changed to refer to the arrangement of the soil mass on bedding, instead of the particles. It is an important factor.

"Porosity" is defined as the equivalent of density or percentage of total pore space. It is an important physical factor, and often determines whether the soil particles are arranged loosely or compactly.

The water in soils may be either of a permanent or of a temporary nature. The permanent water above the water table is due to surface tension and is the capillary water content. Its determination is very important, for it resists the action of gravity and cannot be drained. The constant "dampness" of soils is traceable to this characteristic. The size and arrangement of the pores and the texture of the soil have controlling influence over the quantity of capillary water present. Excess water, or the water of gravitation, is that which responds to the action of gravity and is flowing to a lower level. Such water will vary with the seasons. The phenomena of slides are frequently due to excess water content. Your Committee will continue its study.

REVISED SCHEME OF CLASSIFICATION

Source of Material.—The following sub-divisions of sedentary and transported soils are recognized as representing the first factor in the divisions of soil classification:

Sedentary soils:
- Residual (formed in place);
- Cumulose (accumulated organic matter).

Transported soils:
- Colluvial, or gravity laid;
- Alluvial, or water laid, by streams, lakes, or oceans;
- Aeolian, or wind laid;
- Glacial, or ice laid.

Mineral Composition.—This relates to the evident abundance of minerals in the composition of soils.

Structure.—This refers to the natural occurrence of the soil in beds, masses, pockets, or stratified in layers, and the dip of same, and occasionally by cracks, fissures, etc.

Porosity.—This refers to the arrangement of the particles of the soil. It is the density or percentage of total pore space, and is also an important factor.

REVISED SIZE-GRADES

Separation methods	Size-grades	Diameter of circular openings in millimeters — Range of size-grades	
		Finer than	Coarser than
By count and perforated plate screen.	Stones, coarse	256 0	128 0
	medium	128.0	64.0
	fine	64.0	32.0
	Pebbles, coarse	32 0	16.0
	medium	16.0	8.0
	fine	3.0	4.0
By wire screens	Grits, coarse	4.0	2.0
	medium	2.0	1.0
	fine	1.0	0.5
	Dust, coarse	0.5	0.25
	medium	0.25	0.125
	fine	0.125	0.0625
By electrification	Flour, coarse	0.0625	0.0312
	medium	0.0312	0.0156
	fine	0.0156	0.0078
	Powder, coarse	0.0078	0.0039
	medium	0 0039	0.0019
	fine	0.0019	
By super-centrifuge	Colloids		

Water Content.—This refers to the volume of pore space occupied by water. It is an important physical factor.

Texture.—This refers to the relative range ratio and mid-grain size, as determined from plotting the mass diagram of a granulometric analysis. It is constant and is recognized as an important factor.

Variations in the shape of particles may modify the textural class, such as, flat shaley, slaty, sharp, angular, rounded, corroded-surface grains or fragments of rocks.

The revised size-grades of particles have been tentatively grouped as shown in table.

DEFINITIONS OF SETTLEMENTS, ALLOWABLE LOAD, ETC.

Settlement.—Settlement is defined as the change of horizontal plane of any part, or all, of a structure, occurring after the beginning of construction. Some settlement occurs before the completion of the structure, and some continues for a time after completion. After reaching a state of rest, other work near-by may influence the soil bearing the structure and cause vertical motion again to take place. This may occupy but a short period of time, after which the structure again comes to rest. Such vertical motion, or settlement, is due to a number of causes. The excavation of the soil and the preparation of the bed of the foundation disturbs a portion of the grains at the surface of the soil, and they become somewhat loosely associated. The application of weight to soil in that condition naturally compresses it. The excavation of the soil also removes a considerable weight from the plane at which the foundation is to be started. The removal of that weight permits certain soils to swell or increase in volume due to loss of the restraining influence of the soil itself, thereby becoming less compact than before the excavation. This latter part is negligible from an engineer's point of view.

The building of the foundation has a tendency to compress the soil to, or even exceeding, the degree to which it was originally compressed. The addition of more weight to the foundation than that of the soil removed for its preparation at once increases the compressive stresses, and reduction of volume and consequent settlement take place up to the yield point of the soil for such compression. If the load is increased beyond that limit, displacement of the soil takes place due to the crushing of the grains,

or actual movement of the grains from their original position. All these effects are accompanied by a settlement of the structure. In brief, soil acts in much the same manner as other elastic bodies, within certain limitations.

These different movements or compressions in any soil where it is possible to build without the use of piles, or in other words, where there is not an excess of water causing a more or less viscous condition of the soil, are illustrated by a fairly regular form of compression curve. Such a curve shows a relatively large compression at the beginning of the application of the load, diminishing rapidly as the load is applied, then a considerable increase of weight with a fairly regular compression, until finally rapid break-down occurs as displacement or crushing of the grains of the soil takes place.

Even in wet ground, and using piles, much the same form of compression curve is obtained. There is relatively little resistance at the beginning of the curve, increasing as the soil is compacted and the excess water is driven out. As the grains take an elastic bearing, there is a quite wide range of safe compression. This is lost on overdriving, because actual displacement of a considerable quantity of the surrounding soil results, which is thus somewhat restored to its original condition and further compression is necessary to develop its elastic resistance. The resistance is also often restored by a period of rest, during which there is a readjustment of the grains. Such a readjustment also takes place in the grains in the case where piles are driven by jacks, and it is found that if the jacks are released without restraining the piles, further penetration is necessary to secure the same degree of resistance. All these things point to a high degree of elasticity of the soil considered as a mass.

Settlement, as herein defined, should not be confused with the shrinkage of a mass of soil in a loose state from its own weight or by the action of the weather.

Allowable Load.—The allowable load is, in most cases, determined by the question of settlement and the effect of such settlement on the proposed structure. It is obvious that for certain types of structures the settlement should be kept to an absolute minimum, because such settlement, or at least unequal settlements in different parts of the structure, will cause physical damage. For example: if a highly ornamental cut-stone building settled unequally in different parts, it might be much disfigured

and injured by reason of spalling or cracking. On the other hand, an elevated railway carried by independent piers may settle considerably at one or more of the bearings with no evidence thereof whatever, except as indicated by the grade of the track. Structures on pile foundations, particularly around harbor and river work, almost invariably settle, but usually they are of a type and designed so that a reasonable amount of settlement is not injurious.

It is believed that the limit of allowable load should be based on some definite portion of that part of a compression diagram showing a practically uniform rate of compression with increase of load. The amount of such percentages should vary with the type and importance of the structure. Manifestly, structures such as reservoirs containing fluids, where settlement would cause cracking and leaks, should be limited to a much smaller percentage of the ultimate load than the column piers of an elevated railroad, isolated monuments, etc.

The possibility of water reaching the soil beneath a foundation should receive careful consideration. Certain soils, such as sand and gravel, are not materially affected by saturation, but are readily eroded by flowing water. On the other hand, clay is not eroded by flowing water, but the surface is readily softened and, when so softened, is plastic and flows under pressure.

Subject to the above considerations, it is the opinion of the Committee that, for the greater number of types of structures, the safe bearing value of soil should be limited to one-half the value, shown by a compression diagram, between the point where the soil is merely compacted and that where displacement begins. That value can be modified as needed, either way, depending on the character and importance of the structure proposed. Such a value agrees fairly well with common practice and usual soil conditions as they have been observed.

It should be noted that allowable load, as herein defined, may be regarded as a function of settlement, whereas bearing capacity may be regarded as the ultimate load the soil will bear without displacement, and displacement is the ultimate measure for settlement. With this in view, the Committee proposes to submit to the membership a blank form on which to record observed settlements and loads, with the desire that such records be submitted to it for compilation.

CAUSES OF FAILURE DUE TO SOILS

Your Committee suggests that causes of failure due to soils be divided as indicated by the following tabulation:

Foundations fail and bearing capacity of soils becomes deficient by reason of:

1. Compression.
   - (*a*) From unequal loading within the elastic limit of the soil
   - (*b*) From loss of cohesion
   - (*c*) From crushing edges of the grains
   - (*d*) From shrinkage of organic matter
   - (*e*) From loss of water content
2. Flowing......
   - (*f*) From saturation
   - (*g*) From lack of cohesion under the influence of weight and pressure
   - (*h*) From exceeding cohesive strength
3. Sliding.......
   - (*j*) From sliding of bodies of material on an underlying and usually inclined layer of lubricating material, frequently aided by water
   - (*k*) From sliding of material previously immersed when water level is quickly lowered
   - (*l*) From sliding of a structure on the soil
   - (*m*) From sliding of a structure together with the soil
4. Erosion......
   - (*n*) From flowing water and the fluctuation of the water-table
   - (*o*) From the wind
   - (*p*) From weathering and frost
5. Chemical changes. .....
   - (*q*) From possible chemical influences

1. COMPRESSION

(*a*) *From Unequal Loading within the Elastic Limit of the Soil.*—Practically all soils possess elastic properties, the amount depending on the character and quantity of the cementing material giving it cohesion and the water content. If unequal loads per unit of area are placed on such soil, unequal settlements will take place. The soil, however, will recover on the removal of the loads unless the latter are great enough to overcome its elastic properties, by breaking down its cohesion. Such action is most apparent at some depth beneath the surface, as displace-

ment is retarded by the restraining action of the surrounding soil. This unequal settlement, though slight, may be sufficient with certain soils and structures to cause trouble.

As evidence of such action the rebound, or rising, after releasing the jacks, of piles forced into the soil by direct pressure, may be mentioned. The action is also apparent on releasing jacks and wedges when cribwork is used in temporarily supporting heavy loads, as in the underpinning of buildings, etc.

(*b*) *From Loss of Cohesion.*—The cementing material is that which sometimes binds a sand or gravel and gives it a certain degree of stability as long as the cementing holds, even though such cementing is not sufficiently firm to permit the material to be classed as sandstone, as it would be if the cementing were more secure. Compression of material of this kind corresponds to the crushing of soft rock. It means the destruction of the structure of the cemented material and the conversion of the mass into a granular material without cementation. This change is normally accomplished by a decrease in volume.

. (*c*) *From Crushing Edges of the Grains.*—When a granular material is closely packed, weight is transmitted through it on a bearing surface represented by the narrow edges of many grains. When the pressure is increased, some of these edges are stressed beyond the breaking point and break, and this is accompanied by compression of the material. When the latter is composed of hardgrained silicious materials, the amount of comp·ession under pressures commonly used in engineering works is usually so small that it may be neglected. When, however, the granular material consists, in whole or in part, of grains of softer material, compression in this way may become of practical importance.

(*d*) *From Shrinkage of Organic Matter.*—Materials may be considered as containing from 0 to 100 per cent of organic matter, ranging from clean sands deep below the surface, practically free from organic matter, through loams, poor surface soils, and river silts containing from 2 to 10 per cent or more of organic matter, up to rich surface soils and mucks, and, finally, to peat, which is practically all organic matter. Many of these organic materials are subject to compression under pressure. Such materials are not ordinarily built upon, but they must be included in the classification, for completeness.

(*e*) *From Loss of Water Content.*—This relates to fine-grained materials, containing so much water in their voids that some of it

may be forced out. Under loading such material tends to become compressed, but the rate at which compression takes place is limited by the rate at which the excess quantity of water is able to make its exit, and this depends on the facilities for draining, including the resistance of the material to the passage of water. If the material were clay, so fine in grain as to be perfectly water-tight, the water could never make its exit and the particles would then not be compressible. With a slightly larger grain size the water would be slowly forced out, but it is possible that the rate of exit would be so slow that a gradual settlement might take place, extending even through a long term of years. This condition is found in some hydraulic fills, and it seems to be the most probable explanation of some cases where steady and long continued slow settlement takes place.

However, in undisturbed soils that have been long subjected to natural fluctuations in the water level, a stable condition of grain structure may exist, in which, with light loading, no appreciable compression may appear, yet the loss of water content may be considerable. This natural condition has been found in transported soils along the banks and contiguous to some rivers and tidal streams. It is a structural condition, the existence of which should be proven, and is attributable to the manner of deposition of the soil. If subjected to loading without investigation, it might cause trouble.

2. FLOWING

(*f*) *From Saturation.*—Any granular material temporarily saturated with water may flow. The condition of saturation may be brought about in either of two ways, first by flow of water of sufficient strength through a material to move the grains slightly apart, thus increasing temporarily the volume of the material and making "quicksand" of it; or second, by the sudden compression or movement of material completely saturated with water.

(*g*) *From Lack of Cohesion under the Influence of Weight and Pressure.*—This is the most commonly considered case of flowing. It has been more fully treated and is perhaps better understood than any of the others.

(*h*) *From Exceeding Cohesive Strength.*—This differs from (*g*) in that cohesion plays an important part in limiting the flow. The flow is frequently slow and corresponds to that of an extremely thick or viscous liquid. Such flow ordinarily will not begin until

certain limits of stress have been exceeded for that material, but when once begun, it will ordinarily continue under much lower limits of stress than were necessary to start the motion. In this respect the flow of material results from the breaking down of a certain weak structure due to cohesion or cementing, and is to be compared with the movement previously mentioned under (*b*).

3. SLIDING

(*j*) *From Sliding of Bodies of Material on an Underlying and Usually Inclined Layer of Lubricating Material, Frequently Aided by Water: Particular Case of* (*h*).—This may be illustrated by landslides, such as that of the hillside on which some of the Portland, Oregon, reservoirs are located.

(*k*) *From Sliding of Previously Immersed Material when Water Level is Quickly Lowered: Particular Case of* (*h*).—By floods or a sudden recession of the water level, bodies of saturated materials become unstable and slide when relieved of the balancing water pressure. This has caused trouble in the slopes of dams and natural and artificial waterways.

(*l*) *From Sliding of a Structure on the Soil.*—When such failures take place and reduction of the frictional resistance between the soil and structure has developed, the latter slides on the soil without any disturbance of the underlying material.

(*m*) *From Sliding of a Structure Together with the Soil.*—Extensive areas sometimes move taking with them entire structures without local disturbance due to the structures themselves.

4. EROSION

(*n*) *From Flowing Water and the Fluctuation of the Water-table.*—Examples of trouble by reason of flowing water are evidenced at frequent intervals. Notably in the case of water-front and harbor works when tidal water rises behind such structures and causes erosion at stages of low water, and in the case of structures in or adjacent to streams, by erosion around bridge piers, abutments, etc., requiring rip-rap for their protection.

The flowing of underground water has created sink holes, caverns, fissures, and caused trouble by undiscovered conditions of the soil.

Failures of dams and levees have been caused by the percolating of water, and the consequent erosive action, by transporting the fine material of the foundation beneath the structure through the body of the dam or levee or overtopping the crest.

(*o*) *From the Wind.*—The action of the wind causing erosion of the soil adjacent to structures is evident at many points where such structures are founded on the fine sand along lake or seacoast or desert land.

(*p*) *From Weathering and Frost.*—Erosive action by frost is also apparent in northern latitudes when structures are not founded entirely below the frost range. This causes heaving of the material adjacent to the exposed face, and, on thawing, softening of the material beneath the structure, allowing slight settlement because of the reduced bearing value of the soil.

When the exposed surface of the soil is inclined, the softening action of the sun and the freezing of the water content causes a tendency for the material to pass on down the slope. This action is sometimes apparent in railroad cuts where overhead bridge structures are built to carry highways over the railroads. Unless the foundations for such structures are carried to considerable depths, it is not uncommon after a few years to find that the movement of the soil from frost and sun combined has exposed a considerable portion of the masonry which was originally beneath the ground surface.

5. CHEMICAL CHANGES

(*q*) While chemical changes rarely appear to be the direct cause of soil troubles they are included for the sake of completeness. All movements growing out of changes in the chemical composition of the material, particularly oxidation and hydration of deep-lying materials when exposed to the action of air and water during the progress of the work, may cause trouble.

It is not to be supposed that any strict classification is possible. On the other hand, movement of soils will frequently comprise conditions that would be classified under several of the headings, and there will be gradations between the different classes, so that no strict lines can be drawn. Nevertheless, a classification of the way in which soils move under stress along some such general lines will tend to crystallize views as to the way in which various deformations take place and as to the means for guarding against them. It is especially to be hoped that discussion will

aid in arranging for adequate tests of materials to determine the probability of their movement under the various kinds of stresses that may be brought to act on them, and so connect actual experience with these methods of getting at compressibility, cohesion, and other qualities of materials.

GRAIN SIZE

Previous reports of your Committee refer to the difficulties encountered in laboratory procedure. The subject has involved an original study which has been hindered by lack of facilities and funds. The presentation of uniform methods is under consideration pending discussion of the proposed method of expressing the grain size and degree of mixing of granular materials as found in soils.

Size of Grain.—The size of any particular grain of granular material is defined as the diameter of a sphere of equal volume. The size of grain is always the ultimate standard, and the size of mesh or opening is not to be substituted for it.

When sieves with rectangular or round openings are used, and with all other methods of separation, the method should be tested as far as possible to ascertain the actual size of separation for that particular sieve or procedure, and the result should be stated in grain size rather than in terms of sieve openings.

The method of determining the size of separation of a sieve depends on the fact that, in mechanical tests, toward the end of the operation of sifting, the last particles that pass are nearly all of the same size and approach the size of separation of that sieve.

The most accurate method of determining the diameter of particles in such a small portion of material at the end of sifting is to weigh a counted number of particles and compute the mean diameter from the average weight.

Method of Statement and Plotting.—The cumulative or mass diagram is best used for plotting, and all statements should be made in terms of it. The normal analysis of a granular material may be tabulated under the two headings, namely, size of separation, and percentage by weight of sample finer than size.

The method of reporting the results of analysis as the separate percentages by weight between sieves or between other limits should never be used.

In plotting the results, various kinds of cross-section paper may be used. The matter is not of first importance, but a well

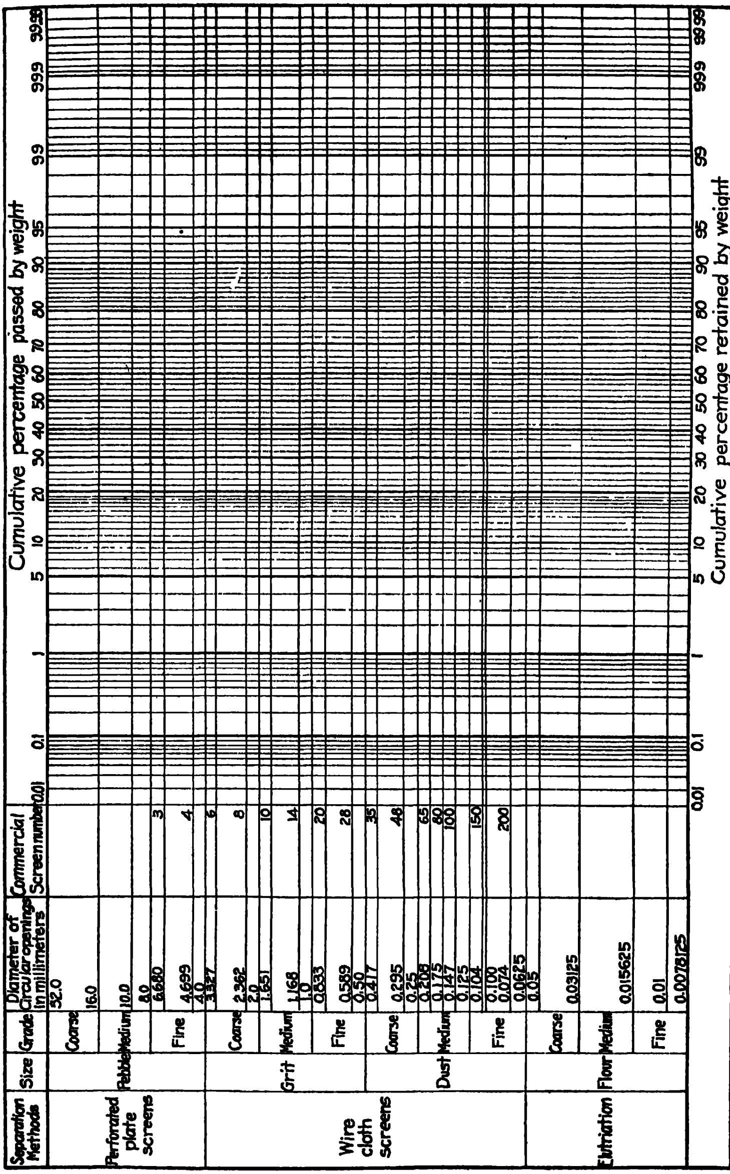

Cumulative percentage passed by weight
Cumulative percentage retained by weight
Separation Methods
Size
Grade
Diameter of Circular openings in millimeters
Commercial Screen number
Perforated plate screens
Wire cloth screens
Elutriation
Pebble
Grit
Dust
Flour
Coarse
Medium
Fine
52.0
16.0
10.0
8.0
6.680
4.699
4.0
3.327
2.362
2.0
1.651
1.168
1.0
0.833
0.589
0.50
0.417
0.295
0.25
0.208
0.175
0.147
0.125
0.104
0.100
0.074
0.0625
0.05
0.03125
0.015625
0.01
0.0078125
3
4
6
8
10
14
20
28
35
48
65
80
100
150
200
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99

Fig. 1.

selected scheme will facilitate plotting and will give equal or greater accuracy with a smaller number of separations. If plotting is made on ordinary cross-section paper, the line bends sharply, and many sieves must be used for full accuracy. If logarithmic paper is used, the number of sieves may be reduced. Logarithmic probability paper seems to be best adapted to plotting mechanical analyses of granular materials, and a good degree of accuracy is obtained with a smaller number of size determinations. For this reason this method of plotting (Fig. 1) is recommended.

Method of Reporting Grain Size.—The mid-grain is defined as that size such that 50 per cent by weight of the sample is finer than it. This is the most generally applicable method of describing the grain size of granular material.

In filtration, the effective size is defined as that size such that 10 per cent of the material is finer than it. This term has an important, though limited, use. It is recognized by the Committee and recommended for use whenever the rate of flow of water through the material must be considered.

Range in Size.—The term "mid-grain size" is of first importance, but it does not give all the information that is needed regarding a granular material. It remains to be stated whether the particles have a small or wide range in size.

The Committee has considered various ways of stating this. It selects as most easily understood and likely to give general satisfaction a statement of the range in size of the grains, this range being selected to include 80 per cent of the particles. Of these particles 10 per cent will be smaller than the smaller limit, and another 10 per cent will be larger than the larger limit.

A complete description of the grain sizes of a common sand may thus be stated:

Mid-grain size. . . . . 0.79 mm.
Range. . . . . 0.33 mm. to 1.90 mm.

The sizes of material that correspond to the various limits are to be taken from a mass diagram made from the analysis figures. It will thus be necessary to plot the figures for each analysis in order to get those which are reported. Plotting is the easiest and most satisfactory way of accomplishing this. It is no hardship to make the plotting for each analysis, as it is easily done, and making it has the advantage that it furnishes a check on the

remainder of the work, for any marked irregularity in the plotted line will tend to attract attention to an erroneous result and lead to its immediate correction.

For many purposes the range only may be stated without the mid-grain size. If this is done, and the mid-grain size is required, it may be taken as the mean proportional of the limits. This is accurate enough for all ordinary purposes.

The range ratio is defined as the ratio between the range limits as just described. It can be obtained directly by dividing the larger figure by the smaller. It will be useful in many comparative studies.

The range ratio is a term that is parallel with the uniformity coefficient which has been used in the analysis of sand for filtration purposes. As it is a parallel term only one should be used. The uniformity coefficient is defined as the ratio between the effective size and that size which is greater than 60 per cent of the grains by weight. The uniformity coefficient is approximately equal to the 0.6 power of the range ratio.

After giving the matter much thought and after considerable discussion by members of the Committee, these methods of defining grain size and range in size are recommended as having a sound and definite basis, and, on the other hand, as being so simple and easily understood as to commend themselves to general use.

MEASURING APPARATUS

Your Committee also presents for your consideration a standard type of a practical field testing apparatus, as shown and fully described by Plates I and II. In designing this apparatus the following points have been kept in mind:

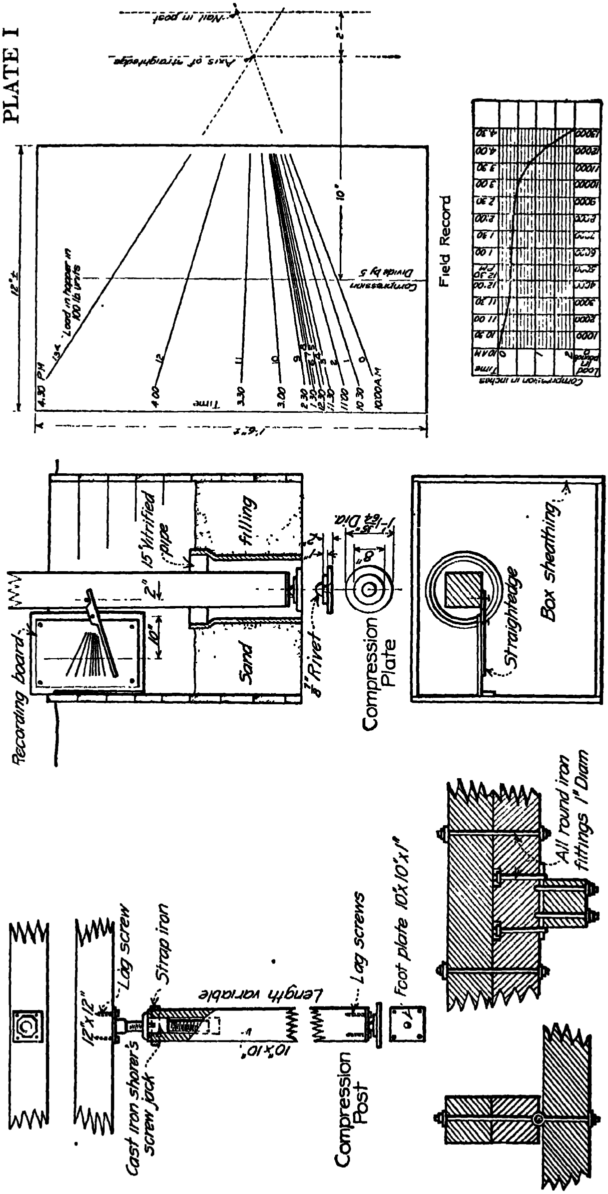
First, a capacity great enough to give a curve extending beyond the limit of safe pressure for ordinary soil or, say, 10 tons per sq. ft.

Second, a degree of sensitiveness such as to give a compression diagram for soils of very light bearing capacity.

Third, a machine requiring for its construction only ordinary labor and materials available on most construction jobs.

Fourth, a machine requiring for the weight to produce compression or pressure only materials such as earth or water, available in any locality where construction is likely to be in progress.

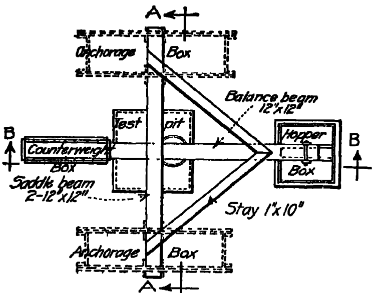
PLATE I



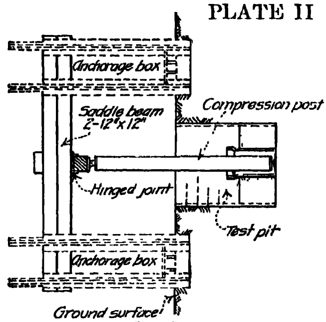
PROPOSED STANDARD LARGE TYPE OF LOAD TESTING APPARATUS FOR SOILS

These imposed conditions at once limit the type of machine, and bring the construction within ordinary carpentry or masonry, or a combination of the two, and limit the substances to be used in producing compression to two, either water or earth.

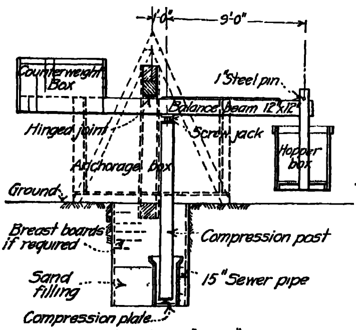
PLATE II



Plan



Section "A-A"



Section "B-B"

NOTE:—Parts shown by dotted lines may be varied at will, to suit local conditions and materials available.

The essential features of this device are as follows: A bearing plate, compression post, adjusting screw, balance beam, ballast hopper, counterweight, saddle beam with anchorages, and the necessary recording device. They should be used in the following manner:

After erecting the anchorage boxes and saddle beam, the boxes should be filled and the balance beam put in position and balanced, with the hopper in place, by filling in the counterweight box. After excavating a test pit (using breasting boards, if necessary), the bottom should be carefully leveled, and the bearing plate placed in position by plumbing down from the plate on the balance beam. The section of sewer pipe should then be placed and carefully backfilled. The compression post may then be placed, the screw adjusted, and the recording device attached. This consists of a board, on which is tacked a sheet of paper, and a light straightedge hinged near the edge of the board. A vertical line is ruled on the paper, at some fixed ratio of the distance from the axis of the straightedge to a nail driven in the compression post. At the start, and as weight is placed in the hopper, lines should be drawn along the straightedge and the time and weight recorded. The balance beam should be kept level, as the post is depressed, by means of the adjusting screw.

The hopper may be calibrated and a scale attached for direct reading of the weight. Either dry sand or water may be used, and provision made near the bottom of the hopper for drawing them off.

All contact points should be coated with heavy grease.

PROPOSED STANDARD LARGE TYPE OF LOAD TESTING APPARATUS FOR SOILS

After considerable thought, the Committee proposes a machine in the form of a balanced scale, built principally of timber. Its capacity is limited to about 10 tons, which is sufficient to cover all cases of ordinary soils, and give a compression diagram exceeding the safe load. In the event that soils of greater or lesser bearing value are encountered, it would be necessary to substitute other bearing plates under the compression post, of smaller or larger areas. With the exception of the bearing plates and the screw at the top of the post for taking up compression, all materials can be shaped and placed in the field, and even these could be prepared in form sufficiently accurate for the purpose, if there is on the work a blacksmith shop and screw-cutting device.

For measuring the amount of compression of the soil, the Committee suggests, as shown by Plate I, a mechanical device, very simple in character, which is preferable to the use of a wire and scale. The use of wire and scale requires that it be constantly watched, and that visual readings of the scale be taken at frequent intervals. The high resistance of most soils makes the reading of the smaller penetrations at different pressures rather uncertain, and a better curve of compression could be obtained by the method suggested. It consists simply of a board, on which is tacked a sheet of paper, with a straight-edge of thin material fastened at a certain fixed distance from a nail on the compression post, against the upper edge of the straight-edge. By drawing a vertical line on the paper, at a definite multiple of the distance between the hinge and nail in the post, a multiplied diagram of the compression for different loads is obtained, reading from a zero line drawn on the paper across the straight-edge before starting the test. At various pressures and compressions, similar lines can be drawn and the time and load noted on the paper, and the actual compression reduced by the ratio of the two distances. By this means a more accurate compression diagram can be made.

Attention is called to the fact that this machine has practically only one moving part, *viz.*, the vertical post causing the compression of the soil. So long as this post maintains a purely vertical position, it is uninfluenced by any distortion of any other part of the machine. From the behavior of timber and soil under such conditions, the Committee is inclined to believe there will be no difficulty in maintaining the post in a vertical

position. It should be borne in mind that this is intended to be a practical field testing machine and no great degree of accuracy should be expected. However, such a machine will take compression tests of soil with a considerable degree of accuracy.

APPENDIX B

Formulas for Bearing Power of Piles

An engineer who is trying to plan a safe and economical pile foundation for any important structure will drive a few test piles at the site and observe their behavior under full load and under heavier loads if possible until they show distinct settlement. A full load test, however, is not practicable for every structure and it is sometimes convenient and useful to have a formula which expresses in a simple and systematic way the relation between the safe bearing capacity of a pile and the variable factors which can be observed during its driving. In this way the results are made available to all who care to apply the formula. A formula does not take the place of loading tests, but the man who takes carefully the necessary measurements and applies them with good judgment in a rational formula, is entitled to express a reasonable prediction as to the bearing power of the piles he is driving.

Engineering News Formulas.—The formulas in general use today in the United States were devised by A. M. Wellington. They are called the Engineering News Formulas because they were first published in that paper on December 29, 1888. Wellington was at that time editor of *Engineering News.* They are incorporated in the building ordinances of some cities and are often used in specifications. The formulas are as follows:

For piles driven by a drop hammer, $P = \frac{2wh}{s+1}$

For piles driven by a steam hammer, $P = \frac{2wh}{s+0.1}$

in which P is the safe load in tons of 2000 lb., w the weight of the hammer in the same unit of weight, h the fall of the hammer in feet, and s the penetration or sinking of pile under the last blow in inches.

This formula is explained by the author as follows:[1]

[1] From "Piles and Pile Driving," by A. M. Wellington, published in 1893, being a reprint of articles which have appeared in *Engineering News* on pile driving and the safe load of piles.

(1) *Bearing Power of Piles Driven by the Method of Ordinary Pile Driving, in which a hammer weighing* 2000 *to* 3000 *lb. or more is dropped* 20 *to* 30 *ft. or more falling free, with an interval of several seconds* (5 *to* 20) *between blows.*

The maximum or ultimate bearing power which is a certainly unsafe load, in the sense that experience shows that piles will rarely bear this load (or any close approach to it) for any length of time without settling, is given by the formula:

$$M = \frac{12wh}{s + 1}$$

in which M = the maximum or ultimate bearing power by any unit of weight.

w = the weight of the hammer by the same unit of weight.

h = the fall of hammer in feet, as below defined and limited.

s = set of pile under last blow in inches as below defined and limited.

1 = a constant which is made necessary by the fact that there is an extra initial resistance in getting a pile under way, and is intended to give the nearest feasible equivalent for the effect of that extra resistance in modifying the mean resistance to penetration. With individual piles it may or may not be a little more or less.

The safe or working load for piles—*i.e.*, the load which it is certainly safe to place upon a pile under all conditions, except as below defined and limited—is shown by experience to be not over one-sixth of the above ultimate load, or

$$\text{Safe load} = \frac{2wh}{s + 1} = \frac{M}{6}$$

in which the symbols have the same meaning as in the prior formula, the application of both of them being subject to the following limiting conditions:

As to w.—The effective weight of the hammer is decreased about 1 per cent. by wind resistance, and perhaps ½ per cent. by guide friction, even when the guides are vertical and in good order. When pile and guides are inclined, the effective weight is decreased: (1) to $h \cos I$ (in which I = the angle of inclination from the vertical) and (2) by the guide friction caused by the force $w \sin I$ pressing the hammer against the guides. With vertical guides this force is theoretically zero.

As to h.—The full fall must only be counted: (1) when there is no sensible bounce after the blow, and (2) when the head of the pile is in good condition. Bouncing in effect divides a single blow into two weaker ones, the energy of the first blow being diminished by an amount of fall equal to the height of the bounce, even if pile and hammer be assumed to be perfectly elastic. As neither is perfectly elastic, at least twice the height of the bounce should be deducted from h to determine its true value for use in the formula.

Condition of the Head.—According to the best existing information a broomed head will destroy from half to three-quarters of the effect of a blow, even if the brooming be only from a half inch to an inch deep. No formula can be safely applied if the last blows be given with the head in such condition; but the remedy is to adze or saw off the heads before giving

the last blows, at least for a few sample piles, and if a very considerable difference is observed, then for all of them, if it is desired to determine and utilize their full bearing power.

As to s.—The proper value can only be determined by taking the mean of the sets for a number of blows, nor then, unless:

(*a*) The penetration has been at a reasonably uniform or uniformly decreasing rate.

(*b*) There is reasonable assurance that the penetration would continue uniform if driven several feet further (which may be known from test piles driven to an extra depth or from general knowledge or evidence as to the nature of the soil, as that it is all sand, gravel or alluvial deposit).

(*c*) The head must be in good condition as noted above.

(*d*) The penetration must be at a reasonably quick as well as uniform rate, not less than ¼ in. for a 3000-lb. hammer falling 30 ft. Any smaller penetrations under such a blow should be assumed to be due to mashing of the point and neglected, and any penetration of less than ½ in. is to be looked on with grave suspicion, and disregarded unless it has been uniform for many blows. With soft wood piles any penetration of less than 1 in. under such a blow is likely to involve destructive strains within the pile as noted below, and hence should be disregarded in computing bearing power.

As to Interval of Time between Blows.—There is nearly always an increase of resistance and decrease of set per blow as an effect of an interval of rest, permitting the earth to settle firmly around the pile. The increase of resistance from a few minutes' to a few hours' rest may vary from 50 per cent to several hundred or even thousand per cent. This effect is usually most pronounced in the finer, soft and wet earths, and least pronounced in coarse sand and gravel. No values of s should therefore be accepted as trustworthy without testing occasional piles for various intervals of rest, and the mean penetration for the first few blows after such an interval of rest should be taken as the value of s.

As to Piles Acting as Columns.—Assuming a blow of $(3000)(20) = 60{,}000$ ft.-lb., a pile which penetrates through soft material to a comparatively hard stratum is not, as a rule, safe as a column (with a factor of safety of 6) for any heavier load than is given by this safe load formula. That is to say for a set of

1 in.,	2 in.,	3 in.,	4 in.,	5 in.,	6 in.

the safe load in pounds by the safe load formula is

60,000	40,000	30,000	24,000	20,000	17,140

which is about one-sixth of the ultimate breaking load of a 10-in. round column of soft wood, of a height of

8 ft.,	14 ft.,	18 ft.,	21 ft.,	24 ft.,	26 ft.

In cases where the length of column without side support is greater than this or the safe load by the safe load formula is less, the safe load by the latter formula will exceed the safe load on the pile as a column.

Crushing Strength.—No pile can be relied on to bear without crushing over 500 to 1000 lb. per sq. in. unless of superior hard wood timber; or 50,000 to 100,000 lb. in all, assuming the average section of the pile to be 100 sq. in.

This is the safe load by the safe load formula for a pile settling 1.4 to 0.2 in. under a 20-ft. blow from a 3000-lb. hammer. Therefore, penetrations of soft wood piles of less than 1½ in. under such a blow (or proportionately for weaker blows) are to be looked on with some suspicion on this account, and penetrations of less than ½ to ¼ in. are to be disregarded wholly in computing bearing power and s taken as $= 0.5$ to 0.25 in.

Uplifting.—Piles driven very close together in certain quicksandy or semi-fluid soils will sometimes rise somewhat when other piles are driven subsequently near them. While the precaution of giving them a few extra setting blows is expedient when time permits, they may be left as they are without much anxiety in most cases, as the phenomenon implies that the lower material is already solidly in contact with the pile, giving as great bearing power as the nature of the soil permits, after the soil has settled solidly about them. It is desirable to avoid this effect if possible, however, which can generally be done by driving the piles butt end down.

Bearing piles should be driven at least 3 ft. center to center each way if this gives a sufficient number to carry the load and they are worse than wasted if driven less than 2 to 2½ ft. center to center.

Variations of Load for Varying Cond tions.—No experimental evidence exists that this safe load formula does not give a safe load under all conditions of service, within the limits of usual values for w, h and s. The load, therefore, need never be made less than the safe load formula permits unless for some particular case of treacherous or dubious soil under an important structure subject to vibratory strains. An extra allowance, if made, should ordinarily be made by reducing the spacing between piles, down to a limit of 2½ ft. center to center. On the other hand, the load should only be made greater than warranted by the safe oad formula with extreme caution, under favorable conditions for high bearing power only, and with care that the requirements as piles acting as columns and as to crushing strength be not infringed.

Computation of Loads.—All extra loads which may result from winds, locomotive counter-weight strains or other temporary loadings are to be considered in computing the load on each pile. In pile trestles, it is none too great an allowance to assume that the entire weight of the driving wheel base falls upon each bent in succession.

(2) *Bearing Power of Piles Driven by Drop Hammer with Hammer Attached to Hoisting Rope.*—When the weight of the hammer has not only to set the hammer in motion but also the hoisting rope and drum, the energy of the blow ($= wh$) is in inverse ratio to the time taken for the hammer to fall a given distance free or attached to the rope, which may be observed experimentally or computed, assuming the mass of the drum to be concentrated at its radius of gyration from its center. It will usually be found to be diminished nearly one-half, which requires a corresponding reduction in the value of h, that variable being supposed to equal the amount of free fall. (Deception is often resorted to in contract work under this method of pile driving; the fall of the hammer being checked by the brake in a way which it is difficult to guard against by inspection. It is therefore a method to be avoided in contract work.)

(3) *Bearing Power of Piles Driven by Water Jet.*—As a rule, the soils in which the water jet works to most advantage are those in which the hammer

method cannot be used at all—*i.e.*, in which the penetration will be little or nothing under any blows which the piles will sustain without crushing. If so the loads which the piles will sustain are limited only by their crushing strength or strength as a column; or the same as if driven by hammer with equally small penetrations. In important cases where there appears room for doubt, the piles should be ested either by loading or preferably by a few blows of a hammer after the material has had a chance to settle firmly about them. In fine river or sea sands, however, it is certain that the penetration would be very small under hammer blows, and hence the bearing power will always be high.

(4) *Bearing Power of Piles Driven by a Dead Load.*—As a rule piles sunk only by a dead load placed on them will not sustain safely much more than the load which sunk them, but will do that (and sometimes much more) after they have stood for a time, to let the material settle closely upon them. In very soft and semi-fluid muds, the safe bearing power may be sometimes the weight which sunk them. The only certain test is to try some of the piles with a hammer after they have been driven some time and then compute by the formula the safe bearing power.

(5) *Bearing Power of Piles Driven by the Steam Hammer.*—As a rule, the interval of time between blows is not more than one-tenth to one-twentieth as great as in ordinary pile driving, and the velocity of impact not over one-third as great. Therefore, the constant 1 in the formula, which represents the extra initial resistance of getting the pile in motion again should not be over one-tenth as great, if so much. Calling t 0.1 for safety we have the case of steam driven piles:

$$\text{Safe load} = \frac{2wh}{s + 0.1}$$

The same results will be reached if we retain the formula for piles driven by a drop hammer but let h = the total fall in feet in ten blows. The formula as thus modified is at least not likely to give excessive loads in either case. If anything, it is somewhat too conservative. There is a lack of experimental data on which to base any closer estimate.

In a discussion of a paper read before the American Society of Civil Engineers by Foster Crowell, Wellington gave the mathematical demonstration of the Engineering News Formula. He made at that time the following general statements concerning it:

This formula was put forward as a purely empirical one, and its usefulness established only by comparing its results with known instances of resistance and with the indications of other formulas, but, as a matter of fact, it was not purely empirical by any means; on the contrary, the general form

$$L = \frac{fwh}{s + c}$$

was first deduced as the theoretically perfect equation of the bearing power of piles barring some trifling and negligible elements to be noted; and I claim in regard to that general form that it includes in proper relation to each other every constant which ought to enter into such a theoretically perfect practical formula, and that it cannot be modified by making it more complex as Mr.

Crowell proposes, or by making it less complex, without making it less trustworthy and correct as well as less convenient. The numerous formulas by such high authorities as Weisbach, Rankine, Sanders, Nystrom and many others, which contain from four to six other constants or variables, and have much more complex forms, I claim to be not only practically unsuitable and for the most part, untrustworthy, but theoretically erroneous; not, of course, in their mathematical work, which I should not venture to question, but in their practical premises. This position I shall not now attempt to establish affirmatively otherwise than by summarizing the process by which my own formula was deduced, which I believe to leave no gaps open for correction by more minute work.

For the complete mathematical demonstration, the reader is referred to the Proceedings of the A. S. C. E. for 1892.

Other Formulas.—The following are the best known of the numerous formulas developed many years ago for the bearing value of piles.[1]

[1] Table taken from pamphlet on "Bearing Piles" by Rudolph Hering.

Authority	Formula for ultimate load	Preferred factor of safety	Extreme load, lb. (2000-lb. hammer falling 30 ft., $s = 0.1$ ft., $p = 500$ lb.)
Sanders	$\frac{wh}{s}$	$\frac{1}{3}$ to $\frac{1}{8}$	600,000
Mason	$\frac{w^2h}{s(w+p)}$	$\frac{1}{4}$	480,000
Trautwine	$60w\sqrt[3]{h}$ (if s is inappreciable)	$\frac{1}{2}$ to $\frac{1}{12}$	372,860
Trautwine	$\frac{5w\sqrt[3]{h}}{s+0.083}$ (if s is appreciable)	$\frac{1}{2}$ to $\frac{1}{12}$	169,790
McAlpine	$80[w + (0.228\sqrt{h} - 1)2240]$	$\frac{1}{3}$ to $\frac{1}{6}$	204,580
Rondelet	427 to 498 lb. per sq. in. (safe load)	$\frac{1}{2}$ to $\frac{1}{3}$	153,720[1] to 179,280[1]
Rankine and Mason	200 lb. per sq. in. (safe load) (pile driven home)	$\frac{1}{2}$ to $\frac{1}{3}$	72,000[1]
Brix and Becker	$\frac{4w^2hp}{s(w+p)^2}$	$\frac{1}{4}$ to $\frac{1}{6}$	384,000
Weisbach	$\frac{w^2h}{s(w+p)} + (w+p)$ when $(w+p)$ is small compared with safe load we have	$\frac{1}{10}$ to $\frac{1}{100}$	482,500
Weisbach and Mason	$\frac{w^2h}{s(w+p)}$ (neglecting weight of pile)	$\frac{1}{10}$ to $\frac{1}{100}$	480,000
Weisbach	$\frac{wh}{s}$	$\frac{1}{10}$ to $\frac{1}{100}$	600,000
Nystrom	$\frac{w^3h}{s(w+p)^2}$	$\frac{1}{6}$	384,000
Brix and Becker	$\frac{w^2hp}{s(w+p)^2}$	$\frac{1}{4}$ to $\frac{1}{6}$	96,000
Engineering News	$\frac{12wh}{s+1}$	$\frac{1}{6}$	327,270

In these formulas:

w = weight of hammer in pounds.

h = fall of hammer in feet.

s = penetration or sinking of pile under last blow in feet. (In the *Engineering News* formula s is expressed in inches.)

p = weight of pile in pounds.

[1] Sectional area assumed 1 sq. ft.

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